California High-Speed Rail Authority

RFP No.: HSR 14-32

Request for Proposals for Design-Build Services for Construction Package 4

Book IV, Part G.1
Geotechnical Baseline Report for Bid (GBR-B)

CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

Preliminary Engineering for Procurement Record Set Submission

Fresno to Bakersfield

Sierra Subdivision
Construction Package 4
Geotechnical Baseline Report for Bid

August 2014

RFP No. HSR 14-32 – INITIAL RELEASE - 05/27/2015

CALIFORNIA
High-Speed Rail Authority

Preliminary Engineering for Procurement
Record Set Submission
Fresno to Bakersfield
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Geotechnical Baseline Report for Bid

Prepared by:

URS/HMM/Arup Joint Venture

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August 2014

This report has been prepared under the direction of the following Geotechnical Engineer.

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August 15, 2014
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List of Abbreviations

AASHTO	American Association of State Highway Transportation Officials
API	American Petroleum Institute
ASTM	ASTM International (formerly American Society for Testing and Materials)
Authority	California High-Speed Rail Authority
bgs	below ground surface
Caltrans	California Department of Transportation
CHSTP	California High-Speed Train Project
CIDH	cast-in-drilled-hole
cm	centimeter
CP	Construction Package
CPT	cone penetration test
E_s	Soil Modulus
FB	Fresno to Bakersfield
ft	Feet
g	gravitational acceleration, 9.81 meters / second^2
GBR-B	Geotechnical Baseline Report for Bid
GBR-C	Geotechnical Baseline Report for Construction
GDR	Geotechnical Data Report
GI	ground investigation
GSHR	Geologic and Seismic Hazards Report
HMM	Hatch Mott MacDonald
HSR	high-speed rail
JPL	Jet Propulsion Laboratories
k_h	modulus of horizontal subgrade reaction
k'_v	modulus of vertical subgrade reaction
kN	kilonewton
MCE	maximum considered earthquake
mi	miles
mm	millimeters
M_W	moment magnitude
N_{60}	standard penetration test N-value corrected for hammer energy
OBE	operating basis earthquake
OSHA	Occupational Safety and Health Administration
ppm	parts per million
PE4P	preliminary engineering for procurement
q_c	CPT tip resistance
q_t	CPT tip resistance corrected for pore water effects
SBT_N	normalized CPT soil behavior type
sec	second
SJV	San Joaquin Valley

SPT	standard penetration test
SR	State Route
T	Period
TM	Technical Memorandum
umhos	Micromhos
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V_s, V_{s30}	shear wave velocity, average shear wave velocity in the upper 30 meters

Section 1.0

Introduction

1.0 Introduction

In 1996, the state of California established the California High-Speed Rail Authority (Authority). The Authority is responsible for studying alternatives to construct a rail system that will provide intercity high-speed rail (HSR) service on over 800 miles of track throughout California. This rail system will connect the major population centers of Sacramento, the San Francisco Bay Area, the Central Valley, Los Angeles, the Inland Empire, Orange County, and San Diego. The Authority is coordinating the project with the Federal Railroad Administration. The California High-Speed Train Project (CHSTP) is envisioned as a state-of-the-art, electrically powered, high-speed, steel-wheel-on-steel-rail technology that will include state-of-the-art safety, signaling, and automated train-control systems.

The statewide CHSTP has been divided into sections for planning, environmental review, coordination, and implementation of the project. This Geotechnical Baseline Report for Bid (GBR-B) is focused on the section of the CHSTP between Fresno and Bakersfield, specifically the Construction Package 4 (CP4), which extends from 1 mile north of the border between Tulare County and Kern County to about 7th Standard Road, north of Bakersfield.

1.1 Geotechnical Contract Documents

The key geotechnical documentation provided in the Contract Documents for CP4 is this Fresno to Bakersfield (FB) CP4 GBR-B. The FB CP4 Geotechnical Data Report (GDR) (URS/HMM/Arup 2014) and the Geologic and Seismic Hazards Report (GSHR) (URS/HMM/Arup 2013c) are also available as reference documents. The CP4 GDR provides details of the ground investigation (GI) such as drilling procedures, soil sampling, in situ testing, hydrogeologic testing, and historical geotechnical information gathered prior to the exploration phase. The CP4 GDR also includes exploration logs, details pertaining to laboratory testing, procedures used to conduct various index tests, strength and deformation tests, and test results. Definitions for terms used in both the CP4 GBR-B and CP4 GDR are contained in Section 11.0, the glossary.

This CP4 GBR-B and the referenced CP4 GDR cover only the FB CP4 corridor.

1.2 Purpose

The principal purpose of this CP4 GBR-B is to set baselines for ground conditions to facilitate the bidding process such that all bidders can rely on a single contractual interpretation of the geotechnical conditions when preparing their bids. This report summarizes anticipated ground conditions for construction of the CP4 alignment, which extends between about 1 mile north of the Tulare/Kern county line and 7th Standard Road, north of Bakersfield.

This CP4 GBR-B is a representation of the conditions upon which the design-build Contractor may rely for bidding. GIs conducted as documented in the CP4 GDR are considered preliminary and shall not be solely relied on for final design. It is incumbent upon the Contractor to conduct supplemental investigations adequate to complete final design and prepare a Geotechnical Baseline Report for Construction (GBR-C). The CP4 GBR-C will serve as the basis of resolution for differing site conditions during construction. The CP4 GBR-B has been prepared such that it will be superseded by the CP4 GBR-C, and the CP4 GBR-C will incorporate additional geotechnical exploration data and analyses. The CP4 GBR-C will become the basis of final design and construction conditions.

The engineering judgment applied in the interpolations and extrapolations of information contained in the CP4 GDR reflect the view of the Authority in establishing the baseline conditions. The baseline conditions for bid presented in this report will (1) serve as a baseline for geotechnical conditions anticipated to be encountered and (2) assist the Contractor in evaluating the requirements for installation of foundation elements and excavating and supporting the ground.

1.3 Report Structure

This report has been prepared in general accordance with Technical Memorandum (TM) 2.9.2 Geotechnical Reports Preparations Guidelines and the latest edition of the American Society of Civil Engineers' publication *Geotechnical Baseline Reports for Construction: Suggested Guidelines* (Essex 2007). Sections 1.0 through 5.0 provide background information, while Sections 6.0 through 9.0 provide specific recommendations related to ground characterization and behavior. Sections 10.0 and 11.0 provide reference information.

Section 1.0 provides an introduction to the project including project location, report purpose, and organization. Section 2.0 provides a project description including key project features and existing man-made structures of significance to the project. Section 3.0 describes sources of geotechnical information including prior geotechnical reports, TMs, data from desk studies, and data from the Preliminary Engineering for Procurement (PE4P) GI for CP4. Section 4.0 describes the project setting through physiography, geology, seismicity, and hydrogeology; Section 5.0 describes previous construction experience in the project vicinity.

Section 6.0 presents ground characterization and geotechnical baselines, Section 7.0 describes design considerations for the various proposed structures, Section 8.0 describes construction considerations, and Section 9.0 discusses instrumentation and monitoring during construction.

Section 10.0 is a list of documents referenced in this report; Section 11.0 is a glossary of terms used in this report.

1.4 Basis of Report

The baseline values in this report have been developed from geotechnical information and data gathered through desk studies and the PE4P CP4 GI, which included widely spaced exploratory boreholes, cone penetration tests (CPTs), and laboratory and field tests. The results from this investigation are presented in the CP4 GDR.

The statements in this document that shall be construed as baselines comprise only those sentences that begin "As a baseline" and "For bidding purposes", or equivalent statements.

All other statements in this document are provided for background and context, or as recommendations and commentary to assist the design-builder's understanding of potential ground-related issues along the alignment. No such statements in this document shall be construed to overrule or supersede any code, regulation, contract requirement, project design criteria, or project specification.

1.5 Project Constraints and Restrictions

The baseline recommendations in this report have been derived from the available data. Limited site access, limited historical data, and wide spacing of explorations constrain the recommendations to a level appropriate for preliminary engineering, not final design. PE4P structures were designed using geotechnical parameters from historical data only. However, when the CP4 GDR and this GBR-B became available, the assumptions made to complete the PE4P structures design using historical data were found to be reasonable when compared to the data collected and baselines developed herein.

During construction, ground behavior will be influenced by the Contractor's selected design, equipment, means, methods, and level of workmanship. The Contractor must assess how these factors will influence ground behavior and baseline values provided in this report in consideration of the project as a whole.

Section 2.0

Project Description

2.0 Project Description

2.1 Fresno to Bakersfield High-Speed Rail Section

The proposed FB Section of the HSR is approximately 114 miles long and traverses a variety of land uses, including farmland, rural communities, small cities and large cities. The FB Section includes viaducts, elevated structures, retaining walls and segments where the HSR will be at-grade or on embankment. The route of the FB Section passes by or through the rural communities of Bowles, Laton, Conejo, Armona, and Allensworth and the cities of Fresno, Hanford, Corcoran, Wasco, Shafter, and Bakersfield.

The FB Section extends from north of Stanislaus Street in Fresno to the northernmost limit of the Bakersfield to Palmdale Section of the HSR at Oswell Street in Bakersfield.

2.2 Alignments

The FB Section is a critical link connecting the northern HSR sections of Merced to Fresno and the Bay Area to the southern HSR sections of Bakersfield to Palmdale and Palmdale to Los Angeles. The FB Section includes HSR stations in the cities of Fresno and Bakersfield, with a third station in the vicinity of Hanford. The Fresno and Bakersfield stations are this section’s project termini.

For the purposes of the environmental document, the FB Section of the HSR was divided into 10 subsections, most of which had multiple alternative alignments. Table 2.2-1 summarizes and Figure 2.2-1 illustrates the subsections and their corresponding alignments. The preferred alternative for CP4 is discussed in Section 2.3.

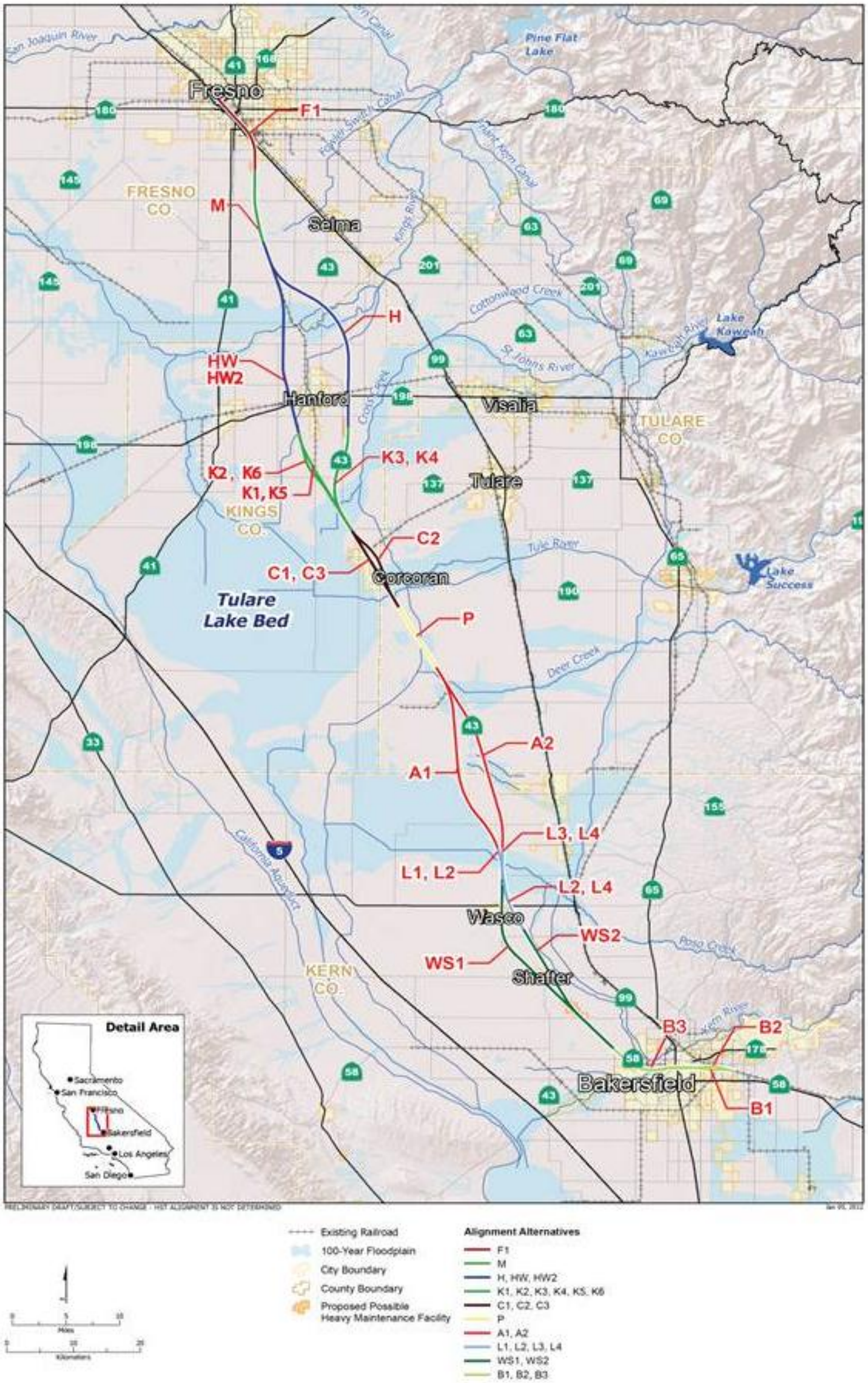

Figure 2.2-1
Overview of Alignment Subsections

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Table 2.2-1
FB Alignment Subsections

<table>
<tr><th rowspan="2">Alignment Prefix</th><th rowspan="2">Alignment Subsection Name</th><th colspan="2">Location</th><th rowspan="2">County</th><th rowspan="2">EIR/EIS Name[a]</th></tr>
<tr><th>Begin</th><th>End</th></tr>
<tr><td>F1</td><td>Fresno</td><td>San Joaquin St</td><td>E Lincoln Ave</td><td>Fresno</td><td>BNSF</td></tr>
<tr><td>M</td><td>Monmouth</td><td>E Lincoln Ave</td><td>E Kamm Ave</td><td>Fresno</td><td>BNSF</td></tr>
<tr><td>H</td><td>Hanford</td><td>E Kamm Ave</td><td>Iona Ave</td><td rowspan="3">Fresno and Kings</td><td>BNSF (Hanford East)</td></tr>
<tr><td>HW</td><td>Hanford West Bypass</td><td>E Kamm Ave</td><td>Idaho Ave</td><td>Hanford West Bypass 1 & 2</td></tr>
<tr><td>HW2</td><td>Hanford West Bypass</td><td>E Kamm Ave</td><td>Iona Ave</td><td>Hanford West Bypass 1 & 2 Modified</td></tr>
<tr><td>K1</td><td rowspan="6">Kaweah</td><td>Idaho Ave</td><td>Nevada Ave</td><td rowspan="6">Kings</td><td>Hanford West Bypass 2 (at-grade) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])</td></tr>
<tr><td>K2</td><td>Idaho Ave</td><td>Nevada Ave</td><td>Hanford West Bypass 1 (at-grade) (connects to C3 [BNSF through Corcoran])</td></tr>
<tr><td>K3</td><td>Iona Ave</td><td>Nevada Ave</td><td>BNSF (Hanford East) (connects to C3 [BNSF through Corcoran])</td></tr>
<tr><td>K4</td><td>Iona Ave</td><td>Nevada Ave</td><td>BNSF (Hanford East) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])</td></tr>
<tr><td>K5</td><td>Iona Ave</td><td>Nevada Ave</td><td>Hanford West Bypass 2 Modified (below-grade) (connects to C1 [Corcoran Elevated] or C2 [Corcoran Bypass])</td></tr>
<tr><td>K6</td><td>Iona Ave</td><td>Nevada Ave</td><td>Hanford West Bypass 1 Modified (below-grade) (connects to C3 [BNSF through Corcoran])</td></tr>
<tr><td>C1</td><td>Corcoran</td><td>Nevada Ave</td><td>Ave 128</td><td rowspan="3">Kings and Tulare</td><td>Corcoran Elevated</td></tr>
<tr><td>C2</td><td>Corcoran Bypass</td><td>Nevada Ave</td><td>Ave 128</td><td>Corcoran Bypass</td></tr>
<tr><td>C3</td><td>Corcoran</td><td>Nevada Ave</td><td>Ave 128</td><td>BNSF (through Corcoran)</td></tr>
<tr><td>P</td><td>Pixley</td><td>Ave 128</td><td>Ave 84</td><td>Tulare</td><td>BNSF</td></tr>
<tr><td>A1</td><td>Allensworth Bypass</td><td>Ave 84</td><td>Elmo Hwy</td><td>Tulare and</td><td>Allensworth Bypass</td></tr>
</table>

Table 2.2-1
FB Alignment Subsections

<table>
<tr><th rowspan="2">Alignment Prefix</th><th rowspan="2">Alignment Subsection Name</th><th colspan="2">Location</th><th rowspan="2">County</th><th rowspan="2">EIR/EIS Name[a]</th></tr>
<tr><th>Begin</th><th>End</th></tr>
<tr><td>A2</td><td>Through Allensworth</td><td>Ave 84</td><td>Elmo Hwy</td><td>Kern</td><td>BNSF (through Allensworth)</td></tr>
<tr><td>L1</td><td rowspan="4">Poso Creek</td><td>Elmo Hwy</td><td>Whisler Rd</td><td rowspan="4">Kern</td><td>Allensworth Bypass (connects to BNSF [through Wasco-Shafter])</td></tr>
<tr><td>L2</td><td>Elmo Hwy</td><td>Poplar Ave</td><td>Allensworth Bypass (connects to Wasco-Shafter Bypass)</td></tr>
<tr><td>L3</td><td>Elmo Hwy</td><td>Whisler Rd</td><td>BNSF (through Allensworth) (connects to BNSF [through Wasco-Shafter])</td></tr>
<tr><td>L4</td><td>Elmo Hwy</td><td>Poplar Ave</td><td>BNSF (through Allensworth) (connects to Wasco-Shafter Bypass)</td></tr>
<tr><td>WS1</td><td>Through Wasco-Shafter</td><td>Whisler Rd</td><td>Hageman Rd</td><td rowspan="2">Kern</td><td>BNSF (through Wasco-Shafter)</td></tr>
<tr><td>WS2</td><td>Wasco-Shafter Bypass</td><td>Poplar Ave</td><td>Hageman Rd</td><td>Wasco-Shafter Bypass</td></tr>
<tr><td>B1</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td rowspan="3">Kern</td><td>BNSF (Bakersfield North)</td></tr>
<tr><td>B2</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td>Bakersfield South</td></tr>
<tr><td>B3</td><td>Bakersfield Urban</td><td>Hageman Rd</td><td>Baker St</td><td>Bakersfield Hybrid</td></tr>
<tr><td colspan="6">[a] Environmental Impact Report/Statement</td></tr>
</table>

2.3 CP4 Alignment Features

The CP4 alignment spans approximately 29 miles, traversing approximately 1 mile of Tulare County through rural farm land to the Kern/Tulare county line then alongside the BNSF railroad and State Route (SR) 43 and Santa Fe Way, through the communities of Wasco and Shafter terminating at 7th Standard Road, north of Bakersfield. Figure 2.3-1 shows the preferred CP4 alignment. The CP4 alignment crosses through rural areas in Tulare County and enters Kern County about 2.7 miles west of SR 43. Heading south into Kern County, the A1 alignment curves to the east and meets SR 43 at about Taussig Ave where A1 becomes the L1 alignment. The L1 alignment continues along the west side of SR 43 and the BNSF railroad until it reaches the north side of Wasco and becomes the WS1 alignment for the remainder of the CP4 subsection. Through Wasco the alignment is on elevated structure/viaduct and retained embankment until it crosses to the east of the BNSF railroad just south of Jackson Avenue, returning to grade and staying approximately parallel to the east side of the BNSF railroad and SR 43. The WS1

alignment rises to an elevated structure as it approaches Shafter just north of Tulare Avenue. Just south of Riverside Street the alignment crosses back to the west side of both the BNSF railroad and SR 43. At Los Angeles Avenue, SR 43 turns south, and the alignment continues parallel to Santa Fe Way, returning to grade south of Burbank Street, and terminates at the intersection of Santa Fe Way with 7th Standard Road, north of Bakersfield.

The CP4 alignment includes at-grade and embankment rail sections as well as retaining walls, bridges and elevated structures. This contract also includes numerous secondary transverse vehicular and pedestrian bridges at select local street intersections. The design requires shallow and deep foundations, retaining walls, and earthwork embankments for the proposed improvements. The key project features are described in Table 2.3-1, from north to south. The table has been populated with the current 15% design structures. Please consult other contract documents for the most updated information.

The CP4 GI, as discussed in the CP4 GDR, focused on the preferred alignment consisting of A1, L1, and WS1 alignments within the limits of CP4, shown in color in Figure 2.3-1.

Table 2.3-1
Summary of Significant Structures in CP4

Structure Type	Approx.Start Station (ft)	Approx.End Station (ft)	Description of Location	Approx. Length (ft)	Structure ID
At-Grade	4435+50	4925+51	From south of Avenue 8 to south of Elmo Highway	49,001	At-Grade 1
At-Grade	5154+50	5191+50	From south of Elmo Highway to south of W Sherwood Ave	3,700	At-Grade 2
Retained Embankment	5191+50	5225+40	From south of W Sherwood Ave to north of Poso Creek	3,390	Retained 1
Structure	5225+40	5227+80	From north of Poso Creek to south of Poso Creek	240	Structure 1
Retained Embankment	5227+80	5271+60	From south of Poso Creek to north of Taussig Ave	4,380	Retained 2
At-Grade	5271+60	5322+33	From north of Taussig Ave to south of Whisler Rd	5,073	At-Grade 3
At-Grade	5422+50	5551+00	From south of Whisler Road to north of Hwy 46	12,850	At-Grade 4
Retained Embankment	5551+00	5556+40	From north of Hwy 46 to north of Hwy 46	540	Retained 3
Structure	5556+40	5557+60	From north of Hwy 46 to south of Hwy 46	120	Structure 2
Retained Embankment	5557+60	5564+80	From south of Hwy 46 to north of 4th St	720	Retained 4
Structure	5564+80	5682+95	From north of 4th Street to north of Prospect Ave	11,815	Structure 3

Table 2.3-1
Summary of Significant Structures in CP4

Structure Type	Approx.Start Station (ft)	Approx.End Station (ft)	Description of Location	Approx. Length (ft)	Structure ID
Retained Embankment	5682+95	5709+50	From north of Prospect Ave to north of Kimberlina Road	2,655	Retained 5
At-Grade	5709+50	5716+02	From north of Kimberlina Rd to Kimberlina Rd	652	At-Grade 5
Structure	5716+02	5716+70	From Kimberlina Rd to south of Kimberlina Rd	68	Structure 4
At-Grade	5716+70	5928+55	From south of Kimberlina Rd to south of W Fresno Ave	21,185	At-Grade 6
Retained Embankment	5928+55	5955+30	From south of W Fresno Ave to north of E Tulare Ave	2,675	Retained 6
Structure	5955+30	6117+25	From north of E Tulare Ave to south of Orange Street	16,195	Structure 5
Retained Embankment	6117+25	6151+00	From south of Orange street to south of Burbank St	3,375	Retained 7
At-Grade	6151+00	6291+00	From south of Burbank St to 7th Standard Rd	14,000	At-Grade 7

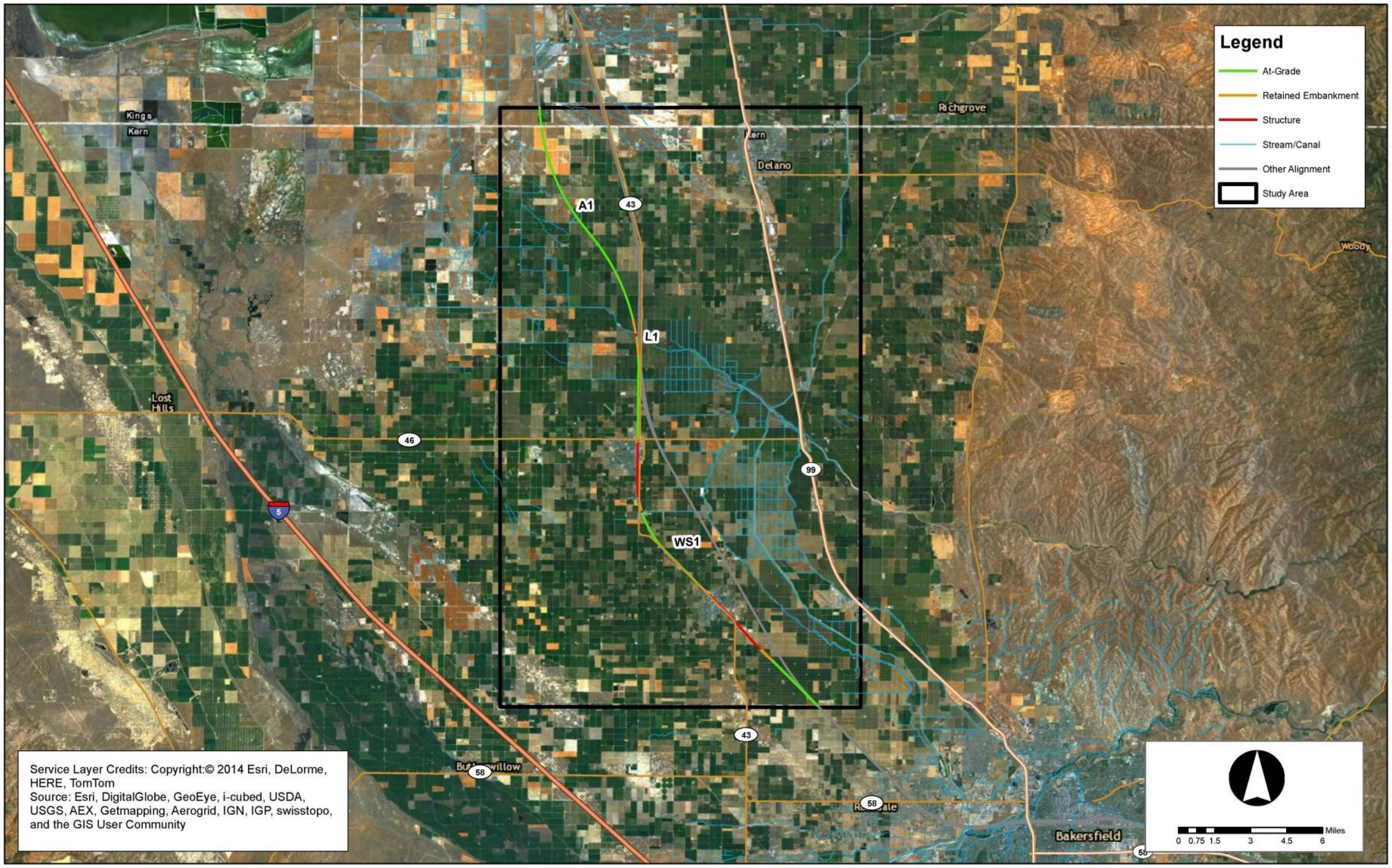

Figure 2.3-1
Vicinity Map of CP4 Alignment

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Section 3.0

Sources of Geologic and Geotechnical Information

3.0 Sources of Geologic and Geotechnical Information

3.1 Project Sources

Data and information for this report were primarily obtained from publically available reports and the results of the PE4P GI. The sources include the following:

- FB Archeological Survey (URS/HMM/Arup 2011).
- FB Geology, Soils, and Seismicity Technical Report (URS/HMM/Arup 2012).
- FB 15% Record Set GI Work Plan (2013a).
- FB Draft Environmental Impact Statement/Report (URS/HMM/Arup 2013b).
- FB 15% Record Set Geologic and Seismic Hazards Report (GSHR; URS/HMM/Arup 2013c).
- FB PE4P Record Set Hydrology, Hydraulics, and Drainage Report (URS/HMM/Arup 2013d).
- FB 15% Record Set Utility Impact Report (URS/HMM/Arup 2013e).
- FB PE4P Record Set CP4 GDR (URS/HMM/Arup 2014).

3.2 Site Investigations

The PE4P GI for CP4 was conducted between August 19 and November 13, 2013, and consisted of drilling 20 rotary-wash boreholes and performing 45 CPTs. Soil samples were collected from boreholes at 5-foot intervals using standard penetration test (SPT) split spoon samplers and California Modified samplers driven with automatic hammers. Energy calibration tests were performed on the automatic hammers used during the exploration program, and SPT N-values were recorded and corrected accordingly. The explorations' names and locations relative to the alignment are presented in Table 3.2-1.

Additional in situ testing performed during the investigation included shear wave velocity (V_s) profiles in four boreholes using the suspension velocity logging method, V_s profiles in six CPTs, and pore water pressure dissipation tests in 43 of 45 CPTs. Four boreholes, S0077R, S0078R, S0083R, and S0088AR, were converted to standpipe piezometers to monitor groundwater-level fluctuations. In situ testing performed during the exploration program also included pocket penetrometer and torvane testing on retrieved samples.

Laboratory testing was performed on representative soil samples to obtain index and engineering properties. Geotechnical index testing included moisture content, density, No. 200 sieve wash, hydrometer, grain-size analysis, specific gravity, Atterberg limits, and organic content tests. Laboratory testing for engineering properties included direct shear, triaxial undrained and drained, compaction, California bearing ratio, and corrosion test methods. Soil corrosivity testing was also performed, including resistivity, pH, sulfate content, and chloride content methods.

Table 3.2-1
Locations of PE4P Ground Investigation Tests Relative to Proposed Alignments

Exploration ID	Alignment Alternative	Structure ID	Distance along CP4, north to south (miles)	Offset Distance from Alignment, (feet)[a]	Elevation (ft) (NAVD 88)
S0243CPT	A1	At-Grade 1	0.83	950	219.3
S0246CPT	A1	At-Grade 1	1.82	-167	220.6
S0249CPT	A1	At-Grade 1	2.75	-1,643	227.2
S0074R	A1	At-Grade 1	2.87	28	229.6
S0248CPT	A1	At-Grade 1	2.93	847	229.5

Table 3.2-1
Locations of PE4P Ground Investigation Tests Relative to Proposed Alignments

Exploratio n ID	Alignment Alternative	Structure ID	Distance along CP4, north to south (miles)	Offset Distance from Alignment, (feet)[a]	Elevation (ft) (NAVD 88)
S0252CPT	A1	At-Grade 1	5.48	2,048	245.3
S0254CPT	A1	At-Grade 1	6.42	-149	257.9
S0075R	A1	At-Grade 1	6.43	-169	257.9
S0076R	A1	At-Grade 1	7.63	-34	269.7
S0257CPT	A1	At-Grade 1	7.63	98	270.0
S0260ACPT	A1	At-Grade 1	8.94	3,163	277.9
S0261CPT	L1	At-Grade 2	9.20	2,484	285.6
S0262CPT	L1	At-Grade 2	9.63	2,025	292.7
S0263CPT	L1	Retained Embankment 1	9.81	1,663	295.1
S0264CPT	L1	Retained Embankment 1	10.12	1,193	299.1
S0077R	L1	Retained Embankment 1	10.28	10	299.0
S0078R	L1	Retained Embankment 2	10.59	693	306.0
S0266CPT	L1	Retained Embankment 2	10.80	578	307.3
S0267CPT	L1	Retained Embankment 2	11.29	-37	304.4
S0079R	L1	At-Grade 3	11.30	-61	304.6
S0270CPT	L1	At-Grade 3	11.53	263	310.2
S0268ACPT	L1	At-Grade 3	11.78	213	310.1
S0080R	WS1	At-Grade 4	12.32	162	312.3
S0269CPT	WS1	At-Grade 4	12.32	214	312.6
S0271CPT	WS1	At-Grade 4	12.78	212	317.6
S0272CPT	WS1	At-Grade 4	13.31	225	320.6
S0081R	WS1	At-Grade 4	13.69	174	320.7
S0273CPT	WS1	At-Grade 4	13.69	229	320.8
S0274CPT	WS1	At-Grade 4	13.80	-1,833	317.6
S0082R	WS1	Retained Embankment 3	14.88	-461	328.3
S0279CPT	WS1	Retained Embankment 3	14.88	-463	328.3
S0280CPT	WS1	Structure 3	15.23	-48	331.4
S0282CPT	WS1	Structure 3	15.48	-36	331.0
S0283CPT	WS1	Structure 3	15.78	-48	332.0
S0083R	WS1	Structure 3	15.79	-93	331.9
S0285ACPT	WS1	Structure 3	16.30	-81	334.0

Table 3.2-1
Locations of PE4P Ground Investigation Tests Relative to Proposed Alignments

Exploration ID	Alignment Alternative	Structure ID	Distance along CP4, north to south (miles)	Offset Distance from Alignment, (feet)[a]	Elevation (ft) (NAVD 88)
S0287CPT	WS1	Structure 3	16.81	356	337.1
S0084R	WS1	Structure 3	16.81	343	337.1
S0289CPT	WS1	Structure 3	17.17	-72	337.2
S0290ACPT	WS1	At-Grade 5	17.79	-36	332.8
S0084AR	WS1	At-Grade 5	17.79	-35	332.8
S0292CPT	WS1	At-Grade 6	18.85	-1,104	346.9
S0085R	WS1	At-Grade 6	20.00	-291	345.0
S0295CPT	WS1	At-Grade 6	20.00	-295	346.9
S0086R	WS1	At-Grade 6	20.98	-296	344.7
S0297CPT	WS1	At-Grade 6	20.99	75	345.9
S0087R	WS1	At-Grade 6	21.67	18	346.6
S0301CPT	WS1	At-Grade 6	21.69	6	346.6
S0302CPT	WS1	Retained Embankment 5	21.94	-206	346.1
S0303CPT	WS1	Retained Embankment 5	22.30	-224	347.6
S0304CPT	WS1	Structure 5	22.75	-33	345.4
S0088R	WS1	Structure 5	23.04	29	344.6
S0305CPT	WS1	Structure 5	23.08	7	344.5
S0308CPT	WS1	Structure 5	23.34	-330	343.7
S0309CPT	WS1	Structure 5	23.77	84	346.1
S0088AR	WS1	Structure 5	24.48	-47	346.1
S0312CPT	WS1	Structure 5	24.48	-33	346.2
S0314CPT	WS1	Structure 5	25.15	-32	343.7
S0315CPT	WS1	Retained Embankment 6	25.58	172	343.4
S0089R	WS1	Retained Embankment 6	25.86	-71	341.8
S0317CPT	WS1	Retained Embankment 6	25.99	133	341.6
S0318ACPT	WS1	At-Grade 7	26.66	60	336.3
S0090R	WS1	At-Grade 7	27.50	21	337.4
S0318CPT	WS1	At-Grade 7	27.88	23	338.7
S0319CPT	WS1	At-Grade 7	28.09	25	339.7
S0091R	WS1	At-Grade 7	28.42	-82	340.4

[a] Positive offsets from the alignment are to the left (generally east) of the alignment with increasing station (progression southward). Negative offsets are to the right of the alignment (generally west).

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3.3 Historical Investigations

The primary source of publicly available historical geotechnical data collected during 15% design was from the California Department of Transportation (Caltrans) database of as-built construction records.

Caltrans data are concentrated along SR 43, 46, and 99, from projects dating between 1952 and 2007. For each project, several boreholes were drilled, logged, and plotted on a cross section. None of the Caltrans records contain laboratory test data. Borehole records collected from Caltrans extend to a maximum depth of 99 feet below ground surface (bgs), with an average borehole depth of 47 feet bgs. Historical Caltrans data are included in Appendix A of the GDR.

In addition, data from registered groundwater wells has been reviewed. Available records provide little information on subsurface conditions. Historical wells are discussed in the GSHR and GDR.

Section 4.0

Physiography and Geology Overview

4.0 Physiography and Geology Overview

This section provides a brief description of physiography, geology, and seismicity within the CP4 corridor. Detailed discussion of physiography, geology, and seismicity along the entire FB alignment is presented in the GSHR.

4.1 Physiography

The CP4 alignment is located within the southern portion of the 450-mile-long Great Valley Geomorphic Province (Bartow 1991). The topography of the Great Valley (the southern portion of which is referred to as the San Joaquin Valley [SJV]) is relatively flat. The SJV is bordered by the Pacific Coast Ranges to the west, the Stockton arch to the north, the Sierra Nevada to the east, and the San Emigdio and Tehachapi mountains to the south.

Superimposed upon this large-scale, relatively flat topographic surface is a localized drainage pattern created by the recent incision of fluvial systems. This localized topography is composed of short, steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees), and narrow gully-type valleys, depending on their age and the amount of flow. Along the CP4 alignment these features appear to have been either channelized or redirected to accommodate the present urbanization.

The topography along the CP4 corridor is generally flat, rising gradually from north to south, and varies between elevations of 219 and 350 feet relative to the North American Vertical Datum of 1988. Localized variations on the ground surface elevation occur at existing road embankments, detention basins, and other man-made features such as irrigation canals and road and rail crossings.

4.2 Geologic Setting

4.2.1 Regional Geology

In his discussion of the geologic evolution of the SJV, Bartow (1991) writes that the SJV is an "asymmetric structural trough that is filled with a prism of upper Mesozoic and Cenozoic sediments up to 9 km [30,000 feet] thick… which at the end of the Mesozoic formed the southern part of an extensive fore-arc basin, evolved during the Cenozoic into today's hybrid intermontane basin."

Bartow (1991) continues discussing the sedimentation infill of the SJV basin, stating that:

> Its evolution comprises the gradual restriction of the marine basin through uplift and emergence of the northern part in the late Paleogene, closing off of the western outlets in the Neogene, and finally the sedimentary infilling in the latest Neogene and Quaternary… these sediments rest on crystalline basement rocks of the southwestward-tilted Sierran block.

4.2.2 Local Geology

Subsurface materials in the vicinity of the CP4 alignment have been generally characterized into four separate map units: (1) existing fill, (2) alluvial fan deposits, (3) basin deposits, and (4) lacustrine deposits. Based on geologic mapping by Smith (1964), Quaternary Pleistocene and Holocene sediments (presumably Sierra Nevada derived), including the fan deposits (Qf), basin deposits (Qb), and Pleistocene non-marine deposits (Qc) are present beneath the CP4 alignment. The geological map indicates the presence of Quaternary lake deposits (Ql) mapped at the

surface within a mile of the north end of the site. There may be significant variability in the horizontal extent of lacustrine deposits with depth and potential for the presence of fine-grained lacustrine deposits beneath portions of the northernmost CP4 alignment.

In the vicinity of the northern CP4 alignment, the Corcoran clay (E-clay) layer of the Lacustrine Tulare Formation is inferred at a depth of approximately 300 feet below ground surface (bgs) by CDWR (1981) and Page (1986). Foster and Saleeby (2003) have since preliminarily mapped the E-clay at or near the surface throughout most of the A1 and L1 sections of the alignment based on interpretation of additional oil and gas well log data. Based on exploration for this project, it does not appear that the E-clay is present in significant contiguous thickness within the upper 100 feet bgs beneath the CP4 alignment. Clay layers of variable thickness are, however, interbedded with coarser sediments in the subsurface profile here. One or more of these layers may represent the margin of the E-clay, which tends to feather out near the lateral extent of lacustrine deposition.

For the majority of the CP4 alignment the depositional environment is dominated by alluvial fan deposits, resulting in interbedded sands, silts, and clays. Between northern Bakersfield and McFarland, several historical east to west trending stream channels exist, associated with the alluvial fan deposits in the area. The channels have been infilled or channelized to facilitate the modern agricultural land. However, relic channel deposits consisting of lenses of clean sand are likely to exist throughout the site.

The continental deposits in the SJV are derived from material from the hills and mountains to the south and east, including the units mapped at the surface along the CP4 alignment, range in total thickness from about 2,300 to greater than 3,000 feet. Underlying these recent alluvial and lacustrine deposits are Pleistocene and Pliocene marine deposits consisting of indurated clays (often referred to as claystones and mudstones) and sands of varying density. Bedrock is believed to be up to approximately 6 miles bgs, becoming shallower with proximity to Bakersfield and the Tehachapi foothills.

This report avoids the use of geologic units in assigning baseline properties because of the potential variability in lateral extent of each unit with depth. Also, different mapped surficial units have generally been found to have similar engineering properties when compared with depth, so drawing distinctions across units is impractical.

4.3 Seismic Setting

The CP4 alignment is located within a relatively seismically quiescent region between the two areas of documented tectonic activity: the Coast Ranges-Sierran Block boundary zone and San Andreas Fault system to the west and the eastern California shear zone to the east.

The Coast Ranges-Sierran Block contains potentially active blind thrust faults (Stein and Eckstrom 1992). The Pacific Coast Ranges contain many active faults that are associated with the northwest-trending San Andreas Fault System, which is the principal tectonic element of the North American-Pacific plate boundary in California. The eastern California shear zone accommodates a portion of the relative movement between the North American and Pacific plates.

4.3.1 Faults and Seismicity

There are no known capable faults crossing or within close proximity to the alignment within the study area. The Pond-Poso Creek Fault is known to cross the CP4 alignment near Woollomes Avenue in Kern County. Although not considered active, the Pond-Poso Creek Fault is a quaternary fault with a structural relationship to the Pond Fault which is classified by CGS as an active fault under the Alquist-Priolo Act. The Pond Fault is about 2 miles to the east of the

alignment. The activity of the Pond Fault has been associated with groundwater extraction; it is not considered seismotectonically active (URS/HMM/Arup JV 2013b and 2013c). Based on the definitions in the TMs, the Pond-Poso Creek Fault could be deemed "capable." However, such a classification mandates a fault rupture analysis, which presumes potential seismic activity. Neither the desk study nor the PE4P GI support the contention that this fault is seismically active. Thus, the Pond-Poso Creek fault is not classified as capable.

No visible surface feature was present along Magnolia Avenue, just southeast of where the Pond-Poso Creek Fault is mapped cross the A1 alignment. However, the GIS locations of faults are known to be off by up to 750 feet (URS/HMM/Arup 2012). Further site-specific investigations using other methods may be warranted to finalize the determination of the capability of the Pond-Poso Fault for final design.

The San Andreas Fault, located approximately 35 miles west of the CP4 alignment, has the highest slip rate and is the most seismically active of any fault near the HSR alignment. The White Wolf Fault is about 30 miles southeast of the alignment, and produced a magnitude 7.5 earthquake when it ruptured in 1952. The San Andreas, White Wolf, Garlock, Kern Canyon, Edison, and Tehachapi Creek Faults and other nearby faults are deemed "capable" by project standards and are described in detail in the FB GSHR (URS/HMM/Arup 2013a).

There are a number of other faults capable of producing large-magnitude earthquakes near the HSR alignment. A list of known faults within 100 miles of the study area and their characteristics are presented in Table 4.3-1.

Table 4.3-1
Characteristics of Faults within 100 Miles of the Study Area (USGS 2006)

Fault Name	Fault Type	Slip Rate (mm/yr)	Distance and Bearing to FB HSR Alignment
San Andreas	Right-Lateral Strike-Slip	20–35	35 miles W of alignment at Wasco
Great Valley (Segments 10–14)	Blind Thrust	1.5	32 miles W of alignment at Wasco
Nunez	–	–	65 miles NW of northern end of alignment
Clovis Fault	–	–	68 miles N of northern end of alignment
Corcoran Clay Fault Zone	Normal	–	N of the alignment from Hanford to the Kern/Tulare County line
Owens Valley	Right-Lateral Strike-Slip	1.5	84 miles NE of alignment
Kern Canyon	Normal	–	55 miles E of northern end of alignment
Kern Front	Normal	–	12 miles E of alignment at Shafter
Kern Gorge	Normal	–	18 miles E of alignment at Shafter
Buena Vista	Thrust	-	23 miles S of southern end of alignment
Southern Sierra Nevada (Independence Section)	Normal	0.1	87 miles NE of northern end of alignment

Table 4.3-1
Characteristics of Faults within 100 Miles of the Study Area (USGS 2006)

Fault Name	Fault Type	Slip Rate (mm/yr)	Distance and Bearing to FB HSR Alignment
Oil Field Fault Zone[a]	Normal	–	30–35 miles E of southern end of alignment
Garlock	Left-Lateral Strike-Slip	2–10	45 miles SE of southern end of alignment
White Wolf	Left-Lateral Reverse	3–8.5	30 miles SE of southern end of alignment
Breckenridge	Normal	–	40 miles E of alignment at Shafter
Poso Creek/Pond	Normal	–	Crosses alignment approximately 3 miles south of border between Tulare and Kern Counties
Wheeler/Pleito	Normal	1.4	30 miles S of southern end of alignment
Edison Fault	Normal	–	22 miles SE of southern end of alignment
Southern Sierra Nevada (Haiwee Reservoir)	Normal	7–14	65 miles E of alignment

[a] These faults appear on the Caltrans 1996 Seismic Hazards Map but have apparently have been de-rated since they do not appear on the Caltrans 2007 Deterministic Peak Ground Acceleration Map.

Source: SCEC 1999, WGCEP 2007, Caltrans 2007, USGS, CGS 2010

4.3.2 Design Earthquake and Design Ground Motion

For the CP4 alignment, two design-level earthquakes have been defined for final design per other contract documents:

> **Maximum considered earthquake (MCE)** – Ground motions corresponding to greater of: (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and; (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to maximum moment magnitude [M_w]) of any fault in the vicinity of the structure.
>
> **Operating basis earthquake (OBE)** – Ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

Site-specific spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield were developed for preliminary engineering. Peak ground accelerations and moment magnitudes used for preliminary liquefaction evaluations are discussed in Section 4.3.3. Acceleration response spectra are provided by the Authority under separate cover.

4.3.3 Liquefaction

Liquefaction assessments for the CP4 alignment were performed for the OBE event using the subsurface data presented in the GDR. CP4 was broken up into three seismic zones, 5, 6 and 7, as provided by the Authority. The analyses were conducted using peak ground accelerations of 0.09g, 0.10g, and 0.11g for seismic zones 5, 6, and 7, respectively. A moment magnitude of 7.9 was used for all analyses. Utilizing the baseline groundwater levels, preliminary evaluations indicate soil liquefaction on a global basis is unlikely to occur during the OBE event on one of the nearby faults; however, localized liquefaction in discrete layers is possible.

For bidding purposes, assume liquefaction will not occur at the OBE; however, the Contractor is required to perform an independent liquefaction hazard analysis for final design.

4.4 Hydrogeologic Setting

4.4.1 Regional

The CP4 HSR alignment is located within the Tule and Kern County Sub-basins (CDWR 1980). A hydrogeologic cross section of the basin is included in the CP4 GDR. Groundwater within this basin is managed by multiple stakeholders. Groundwater is the sole source of drinking water in the region. The current and potential uses of groundwater in the basin are municipal and domestic supply, industrial process supply, industrial service water supply, and agricultural and livestock water supply.

The regional groundwater flow direction in Kern County is from east to west. There are some localized influences as a result of pumping, surface water treatment, and groundwater recharge appurtenances.

4.4.2 Major Aquifers

The depositional environment has formed a sequence of aquifers and aquitards that vary in thickness and lateral continuity. Aquifers are generally composed of granular water-bearing sediments, and aquitards are composed of finer-grained sediments that retard water flow. Most of the aquifers underlying the study area are unconfined but can be semiconfined in isolated locations.

Generally, there are no extensive, low-permeability soils that isolate the upper aquifers from the lower aquifers. The Corcoran Clay (E-Clay) and correlative layers have been mapped beneath the northern portions of the CP4 alignment near the Tulare/Kern County border at depths of less than 100 feet bgs (Foster and Saleeby 2003). The southern extent of the Corcoran Clay has been mapped north of Wasco. However, in this area it is believed that the Corcoran Clay transitions to silty/sandy loam.

4.4.3 Current Groundwater Conditions

Groundwater levels were monitored as part of the PE4P CP4 GI (refer to Section 3.2 and 6.3). The measured groundwater levels at the Tulare/Kern County border are shallow (typically 20 to 50 feet bgs) but become deeper progressing south along the alignment. As described in the CP4 GDR (URS/HMM/Arup 2014), the depth to current groundwater levels in Kern County generally increases to the south and varies from 50 to 125 feet bgs.

Perched groundwater was typically encountered at shallow depths between 5 to 15 feet bgs in Tulare and Kern Counties. Isolated perched groundwater was encountered at depths between 120 and 128 feet bgs, and again at 140 and 148 feet bgs in S0088R. It is anticipated that perched groundwater will likely be encountered during construction. A 1966 soil survey for Kings

County prepared by the U.S. Department of Agriculture's Soil Conservation Service (USDA 2008) indicates that perched groundwater shallower than 6 feet may be present south of Cross Creek near Kansas Avenue.

Further discussion of perched groundwater conditions is included in Section 8.6. Baseline groundwater levels are presented in Section 6.3.

4.4.4 Land Subsidence

Many areas within the SJV have experienced significant subsidence due to groundwater extraction. The southern SJV has been the subject of an extensive investigation between 2007 and 2011 conducted by the Jet Propulsion Laboratories (JPL) (Farr and Liu 2014) using remote sensing technology. The GDR (URS/HMM/Arup 2014) includes the results of a cursory assessment of land subsidence made within the limits of CP4 by JPL. The JPL subsidence rate evaluation indicates that a significant subsidence bowl has developed between Hanford and Allensworth. The CP4 alignment begins at the southern fringes of this bowl. JPL has measured a subsidence rate of about 3 centimeters/year (1.2 inches/year) for the portion of the CP4 alignment extending from the northern terminus to Pond Road. Further south, in Kern County, Lofgren and Klausing (1969) indicate that the area between the border of Tulare and Kern Counties and Wasco experienced subsidence on the order of 2 feet between 1948 and 1962. It is possible that continued subsidence in this area has occurred in the intervening timeframe.

Section 5.0

Related Construction

5.0 Related Construction

The following is a brief description of several large, transportation-related infrastructure improvements in the vicinity of the proposed CP4 alignment from which some GI data have been obtained. These data provide some insight on large infrastructure construction in the vicinity of CP4. Three freeways of the California State Highway System either traverse or are adjacent to the proposed alignment.

The alignment crosses over SR 46 in Kern County at the northern end of Wasco. Historical borings provided by Caltrans can be found at the Calloway Canal and Friant Kern Canal Crossing at about 3.9 and 5.2 miles east of the HSR alignment.

SR 43 is within about 3 miles of the alignment within Tulare County and adjacent to the alignment within Kern County. The BNSF Railway is adjacent to SR 43 through Tulare and Kern Counties until E Los Angeles Avenue where SR 43 heads due south and the HSR and BNSF alignments traverse to the southwest into Bakersfield. The only historic borings available along SR 43 in this reach of the alignment are found at Poso Creek.

SR 99 is a four-lane divided highway. In Tulare, it is about 8.5 miles east of the alignment. In Kern County, SR 99 approaches within 4.25 miles of the alignment. Structures along SR 99 for which historic boring information can be found are located in Delano, Poso Creek, and at SR 46. However, the closest of these are still 6.85 miles from the HSR alignment.

Geotechnical logs of test borings and as-built drawings for several overpasses and bridges along these freeways were collected from a Caltrans database. These logs of test borings are presented in Appendix A of the FB CP4 GDR. Additional information regarding construction methods, ground behavior, groundwater conditions, ground support methods, and problems during construction was not provided in the as-built construction records obtained from Caltrans.

Information was not available from the adjacent railroads.

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Section 6.0

Ground Characterization

6.0 Ground Characterization

6.1 Overview

6.1.1 Organization

Section 6.1 presents the results of subsurface interpretations undertaken to explore the spatial distribution of soil conditions for CP4. The variation of soil type and properties along the alignment and with depth was analyzed to identify trends that support the sub dividing of the subsurface conditions into zones that may warrant separate geotechnical baselines.

Baseline groundwater conditions for design and construction are presented in Section 6.3, followed by additional sections on contamination and corrosion potential. A brief discussion of the scope of the investigation to address contaminated ground and corrosivity is presented in Section 6.4 and 6.5, respectively. Baseline engineering parameters are presented in Section 6.6 for coarse- and fine-grained soil types, as well as by depth, as noted.

Section 6.7 provides discussion and baseline statements regarding soil behaviors, such as long-term settlements, with special relevance to the design and/or construction of the proposed works. Further discussion of design and construction considerations is presented in subsequent sections (Sections 7.0 and 8.0), including further baseline statements.

6.1.2 Subsurface Description

Subsurface conditions comprise interbedded coarse- and fine-grained soils representative of predominately Quaternary basin and alluvial fan deposits of the Great Valley Sequence and of Pleistocene non-marine deposits. From north to south, the CP4 alignment appears to transition from lacustrine and basin deposits to alluvial fan deposits. This transition coincides with a rise in ground surface elevation. The basin and alluvial fan deposits are interbedded in nature, alternating between predominately coarse- and predominately fine-grained material, with more variability in the uppermost 20 feet. Baselines in the form of percent distributions of coarse- and fine-grained soils above and below 20-foot depth are provided.

Coarse-grained soils were observed to vary from loose to dense at shallow depths, increasing to medium dense to very dense at greater depths. Loose and medium-dense sand deposits are substantially concentrated above depths of 20 feet, with the majority of loose sand encountered within the uppermost 12 feet and more commonly at the southern end of CP4 beginning 5 miles south of the start of alignment WS1.

Fine-grained soils were observed to vary from medium-stiff to hard, generally increasing in stiffness with depth. The existence of medium-stiff fines is uncommon and isolated to the uppermost 40 feet bgs. Deeper than 40 feet bgs, fine-grained soils comprise generally very stiff to hard silt or clay.

As the extent of GI was limited, it is possible that actual conditions may vary and differ from the data collected. Furthermore, data pertaining to near-surface soils (defined herein as being within 5 feet bgs) are particularly limited. CPTs and borings were hand-augered to 5 feet depth to clear potential utility conflicts. Test pits were not undertaken due to site access restrictions. Hand augering was used to collect bulk samples for earthwork testing.

6.2 Baseline Description of Subsurface Conditions

6.2.1 Existing Fill and Near-Surface Soil

Fill comprises soils artificially placed, and is commonly encountered near built-up land such as roadway embankments. Fill is occasionally combined with foreign matter such as brick or other man-made debris. The soils that are used may originate from local or distant borrow sources and, in the case of the former, can be difficult to discern from the underlying native soil.

In general, for access reasons, boreholes from the PE4P GI were undertaken adjacent to existing roads. These roads were primarily at-grade or near-grade, and exploratory boreholes penetrated variable depths of embankment fill which would not necessarily be indicative of fill depths outside of the roadway boundary.

Existing fill was noted in 10 of the 20 PE4P boreholes. In two locations, fill was noted to depths greater than 10 feet: S0078R to 18 feet and S0091R to 13 feet. S0078R was located in a median area between existing rail and SR 43, near an approach to a crossing. The location of this borehole is offset over 600 feet from the preferred L1 alignment, and as such will not accurately reflect shallow subsurface conditions of the alignment. The fill encountered, however, may be loosely representative of possible fill material in other locations where roadways intersect the alignment. For instance, S0091R was located on a roadway embankment along Galpin Street approaching Santa Fe Highway, near the footprint of the alignment.

Near-surface soil not explicitly identified as existing fill in the logs may nonetheless contain shallow depths of man-made ground. Regardless of origin, the character of in situ near-surface soil is of interest to the proposed earthworks as this material is often removed or reworked for engineering purposes. An appreciation of the distribution of near-surface soil types encountered within the uppermost 5 feet of all boreholes (or deeper, if Fill was identified) is provided in Table 6.2-1. CPTs were predrilled past 5 feet, and no soil descriptions are available.

Earthwork testing was limited to bulk samples comprising the uppermost 5 feet of drilled boreholes, and engineering baselines are provided in Section 6.6. Comparison of the soil types tested for compaction to those suggested by the limited data below provide an indication of the general applicability of the results.

Table 6.2-1
USCS Distribution of Existing Fill by Percentage of Depth Explored

Borehole ID	Depth (ft)	ML	CL	ML/ SM	SC	SM/ ML	SP	SM	GP
S0074R	5.0[f]	-	-	10.0%	60.0%	-	-	30.0%	-
S0075R	5.0	50.0	-	50.0%	-	-	-	-	-
S0076R	5.0	-	-	-	-	-	-	100%	-
S0077R	5.0	100%	-	-	-	-	-	-	-
S0078R	18.0[Fill]	2.8%	-	-	25.0%	11.1%	55.6%	-	5.6%
S0079R	5.0	100%	-	-	-	-	-	-	-
S0080R	7.5[Fill]	-	33.3%	-	-	-	-	66.7%	-

Table 6.2-1
USCS Distribution of Existing Fill by Percentage of Depth Explored

Borehole ID	Depth (ft)	ML	CL	ML/ SM	SC	SM/ ML	SP	SM	GP
S0081R	5.0[f]	-	-	-	-	-	30.0%	70.0%	-
S0082R	5.0	-	-	-	-	-	-	100%	-
S0083R	8.0[Fill]	-	-	-	-	-	-	100%	-
S0084AR	5.0	-	-	-	-	-	-	100%	-
S0084R	5.0[f]	-	-	-	-	-	-	100%	-
S0085R	5.0[f]	-	-	-	-	-	-	100%	-
S0086R	5.0	-	-	-	-	-	-	100%	-
S0087R	5.0	-	-	-	-	-	-	100%	-
S0088AR	8.0[Fill]	-	-	-	-	-	-	100%	-
S0088R	5.0	100%	-	-	-	-	-	-	-
S0089R	5.0[f]	-	-	-	-	-	-	100%	-
S0090R	5.0	-	-	-	-	-	-	100%	-
S0091R	13.0[Fill]	-	-	-	38.5%	-	-	61.5%	-

[f] Near-surface zone (5 feet bgs) contains existing fill, as noted in borehole log
[Fill] Depth of existing fill exceeds near-surface zone to the depth noted, as per borehole log

A total of 11 grain size distribution tests have been undertaken on samples of existing fill or near-surface soils. The results of this testing is presented in Figure 6.2-4.

Existing fill can also include surface pavements consisting of asphalt concrete (AC), concrete, and aggregate base. Where present (based on the design-builder's review of reference drawings and existing conditions), for bidding purposes, assume existing AC or concrete pavements are 3 inches thick on minor roads and 8 inches thick on improved sections of SR 43 or other highways. For bidding purposes, further assume minor roads have 6 inches of gravel or aggregate base underlying the AC and the highways have 12 inches of aggregate base underlying the AC. Do not assume the existing aggregate base can be directly reused as aggregate base.

The nature of drilling and sampling methods used and spacing of boreholes makes it difficult to quantify the maximum size of fragments in existing fill. For bidding purposes, assume debris up to 1 foot in greatest dimension are present in existing fill. Debris most commonly pertains to rock fragments but may also include rubbish, rubble, or remnants of previous development.

Insufficient data are available to develop baseline parameters of soil laden with organics or disturbed from previous site uses (such as farm fields, orchards, or existing development).

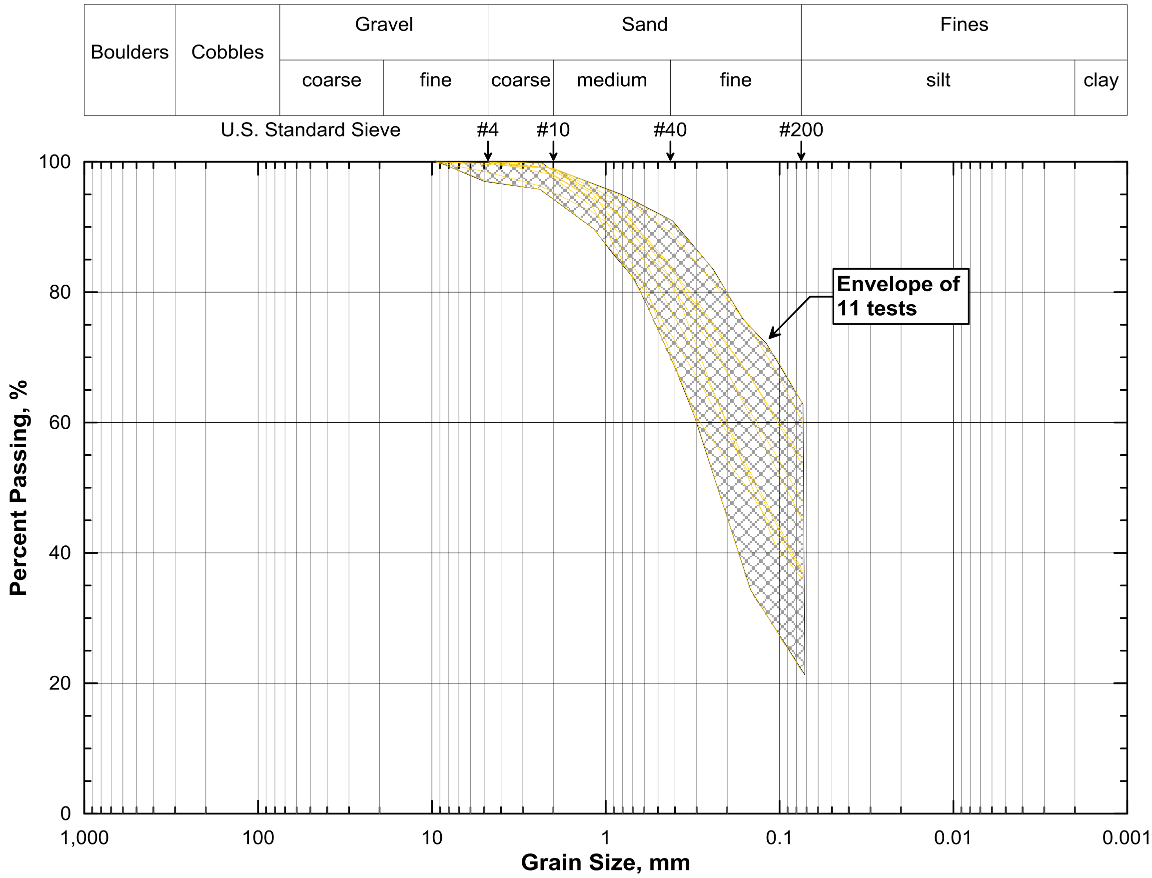


Figure 6.2-1
Grain Size Distribution of Fill and Near-Surface Soils Encountered

6.2.2 Native Soils

The native soils underlying the existing fill and near-surface soil at the locations explored in the CP4 alignments predominantly comprise interbedded layers of sand, silt, and clay. Layers of native soils have been classified predominantly as poorly graded sand with variable silt (SP and SP-SM), silty sand (SM), clayey sand (SC), sand/silt (SM/ML), silt/sand (ML/SM), sandy silt to silt (ML), clayey silt to silty clay (CL-ML), and sandy lean clay to lean clay (CL). Well-graded sand with variable silt (SW and SW-SM) was encountered sporadically, as well as a single observation of sandy fat clay (CH) from 8 to 13 feet depth in S0088R.

Assessment of SPT N and q_c indicates that coarse-grained soils are generally loose to very dense and the fine-grained soils are generally medium stiff to hard.

The distribution of USCS soil type by borehole as a percent of total depth explored is provided in Table 6.2-2. Soil types have been further combined into more general coarse-grained and fine-grained categories, as defined in Table 6.2-2 and further discussed later in this section.

Table 6.2-2
USCS Distribution of Soils by Boring, as a Percentage of Depth Explored

Borehole ID	Fine-Grained %									Coarse-Grained %									
	Total	ML	CL	MH	CH	CL-ML	CL-ML/SM	CL/SC	ML/SM	Total	SC-CL	SC	SM/ML	SW-SM	SP	SP-SM	SM	SW	GP
S0074AR	56.8	38.3	14.8	-	-	-	-	-	3.8	43.2	-	19.7	-	-	-	-	23.5	-	-
S0075R	51.5	44.2	-	-	-	-	-	-	7.4	48.5	-	-	-	-	6.1	10.4	31.9	-	-
S0076R	17.2	6.9	7.9	-	-	2.5	-	-	-	82.8	-	4.4	-	-	11.8	8.9	57.6	-	-
S0077R	62.1	23.6	31.8	-	-	-	-	3.6	3.0	37.9	-	2.1	-	-	2.1	9.4	24.2	-	-
S0078R	40.1	13.6	24.2	-	-	-	-	-	2.3	59.9	-	10.9	1.3	-	8.6	4.6	33.8	-	0.7
S0079R	62.6	19.2	16.3	-	-	-	-	-	27.1	37.4	-	6.4	6.9	-	9.9	-	14.3	-	-
S0080R	37.4	18.4	3.1	-	-	4.3	-	-	11.7	62.6	-	-	-	-	18.4	6.1	38.0	-	-
S0081R	24.5	12.3	6.1	-	-	-	-	-	6.1	75.5	-	-	5.5	-	16.0	12.3	41.7	-	-
S0082R	27.8	8.2	5.4	-	-	14.2	-	-	-	72.2	-	4.4	3.2	-	-	13.6	50.9	-	-
S0083R	26.7	1.3	12.2	-	-	8.9	4.3	-	-	73.3	-	7.3	-	2.3	3.3	19.1	41.3	-	-
S0084AR	27.4	14.1	9.5	-	-	3.8	-	-	-	72.6	-	-	-	3.8	2.7	11.4	54.8	-	-
S0085R	13.3	2.3	7.2	-	-	3.8	-	-	-	86.7	5.3	2.7	-	3.8	1.9	16.3	56.7	-	-
S0086R	42.4	8.4	24.1	-	-	9.9	-	-	-	57.6	-	18.2	-	-	6.9	9.9	22.7	-	-
S0087R	36.1	9.9	8.4	-	-	10.4	3.0	-	4.5	63.9	-	5.0	5.0	-	2.0	7.9	44.1	-	-
S0088R	37.8	19.4	12.8	-	-	5.6	-	-	-	62.2	-	16.7	-	-	2.8	-	42.8	-	-
S0088AR	42.4	15.2	23.8	-	-	3.3	-	-	-	57.6	-	3.3	-	-	12.3	5.3	36.8	-	-
S0089R	38.5	20.3	3.3	-	3.0	8.2	-	-	3.6	61.5	-	-	4.8	-	-	3.0	53.6	-	-
S0090R	46.0	27.6	14.1	-	-	4.3	-	-	-	54.0	-	-	-	-	6.7	6.1	41.1	-	-
S0091R	27.5	14.3	9.2	-	-	-	-	4.0	-	72.5	-	8.1	-	-	28.2	2.9	30.0	3.3	-

A histogram depicting the distribution of USCS soil types is provided in Figure 6.2-2. Soil type can also be inferred from CPT data using Soil Behavior Type (SBT) methods proposed by Robertson (1990), and a histogram for Normalized Soil Behavior Type (SBT_N) distribution is provided in Figure 6.2-3. Both histograms suggest variable ground, with silty sand (to sandy silt) occurring with greatest frequency for the depths explored along the CP4 alignments.

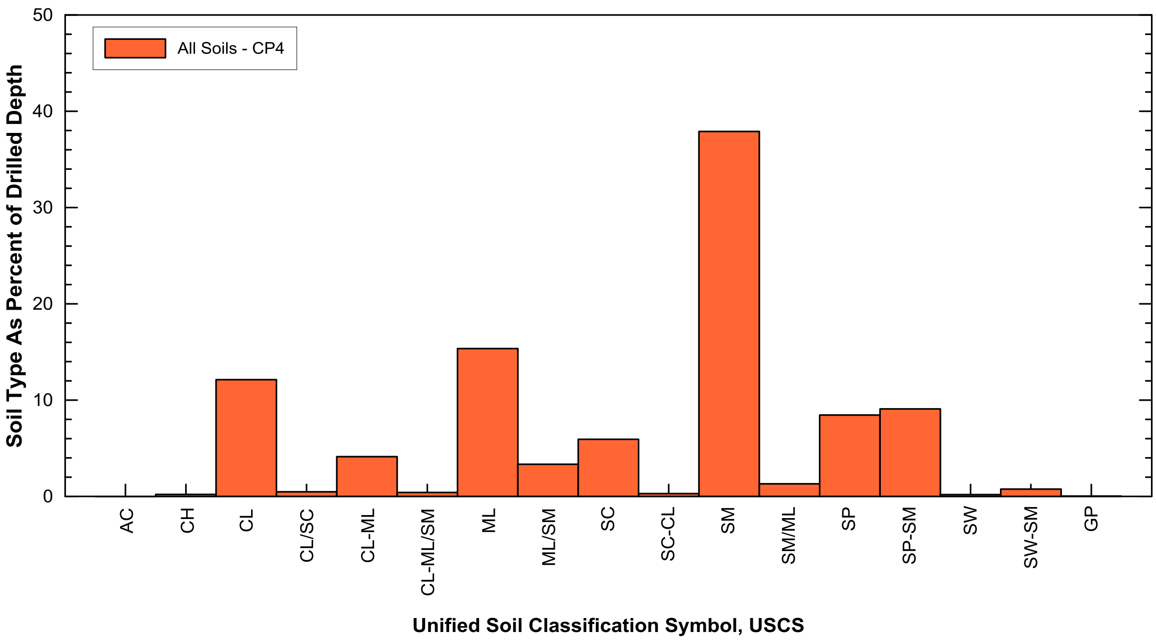
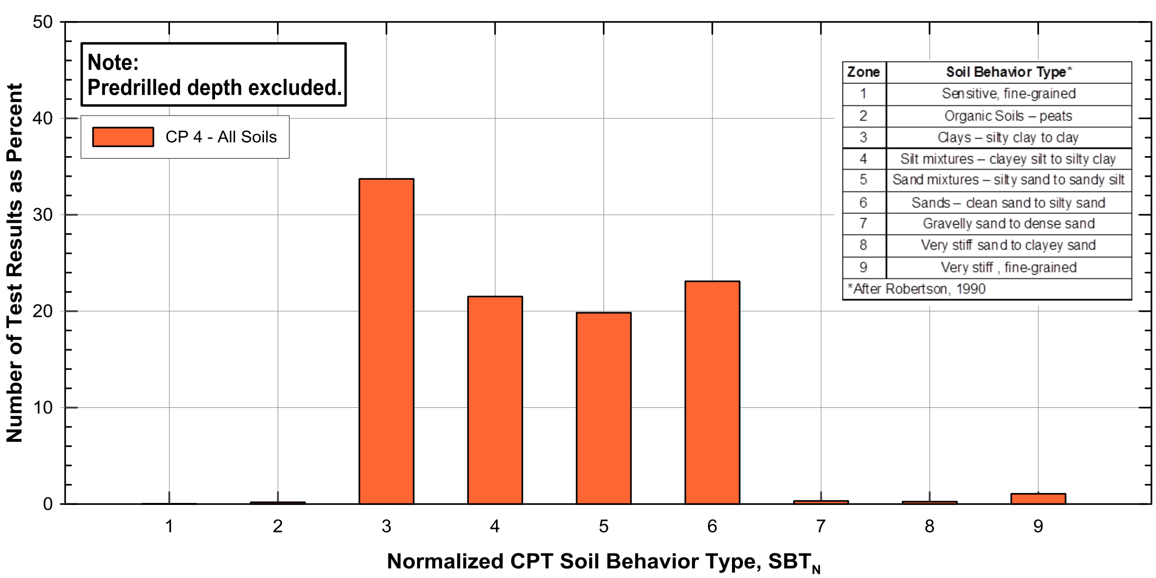


Figure 6.2-2
USCS Distribution for All Soils



Zone	Soil Behavior Type*
1	Sensitive, fine-grained
2	Organic Soils – peats
3	Clays – silty clay to clay
4	Silt mixtures – clayey silt to silty clay
5	Sand mixtures – silty sand to sandy silt
6	Sands – clean sand to silty sand
7	Gravelly sand to dense sand
8	Very stiff sand to clayey sand
9	Very stiff , fine-grained

*After Robertson, 1990

Figure 6.2-3
SBT_N Distribution for All Soils

Grain size distribution curves for 149 tested samples in native soils are presented in Figure 6.2-4. These curves represent the results of laboratory sieve and hydrometer testing performed on

samples of soil from boreholes drilled during the PE4P investigation. The frequency of gradation tests with depth are shown in Figure 6.2-5.

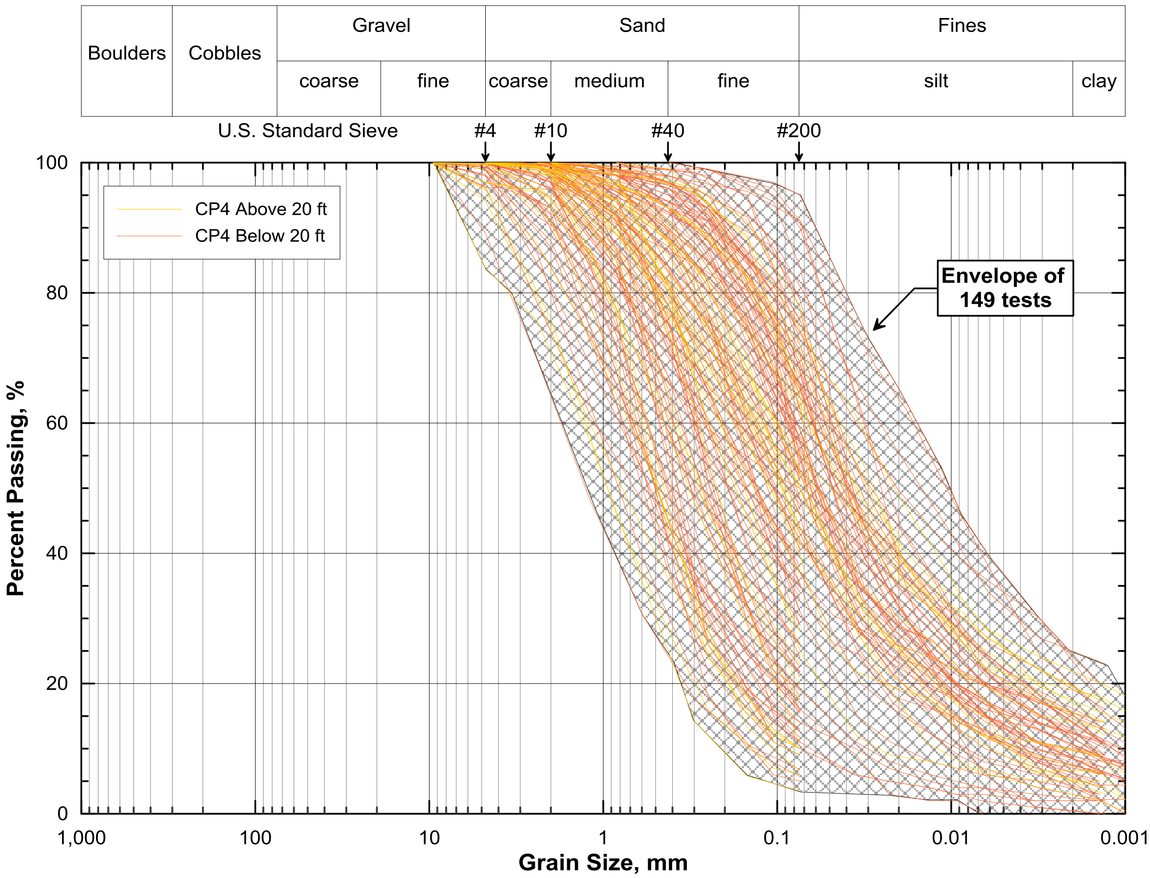


Figure 6.2-4
Grain Size Distribution of Native Soils Encountered

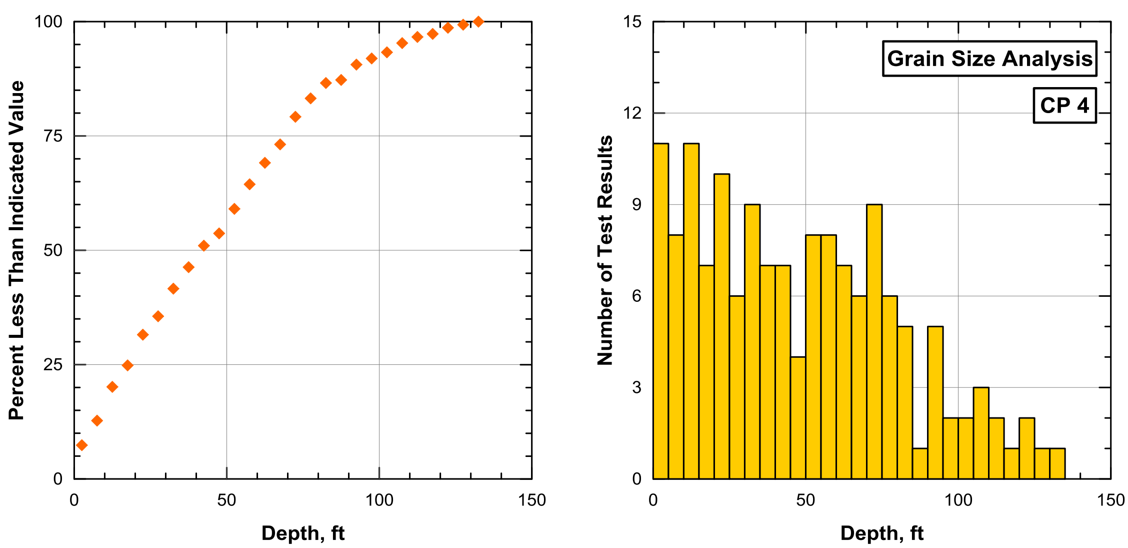


Figure 6.2-5
Probability and Frequency Distribution of Grain Size Analysis with Depth

Atterberg limits tests were carried out on 49 samples, and the distribution of plasticity results are presented in Figure 6.2-6. The probability distribution of Atterberg limits tests with depth is presented in Figure 6.2-7. All tested materials were inorganic and plotted with USCS identifications of predominantly clay (CL) and clayey silt (CL-ML) to silt (ML). High-plasticity clay (CH) was only encountered in one shallow sample in S0088R.

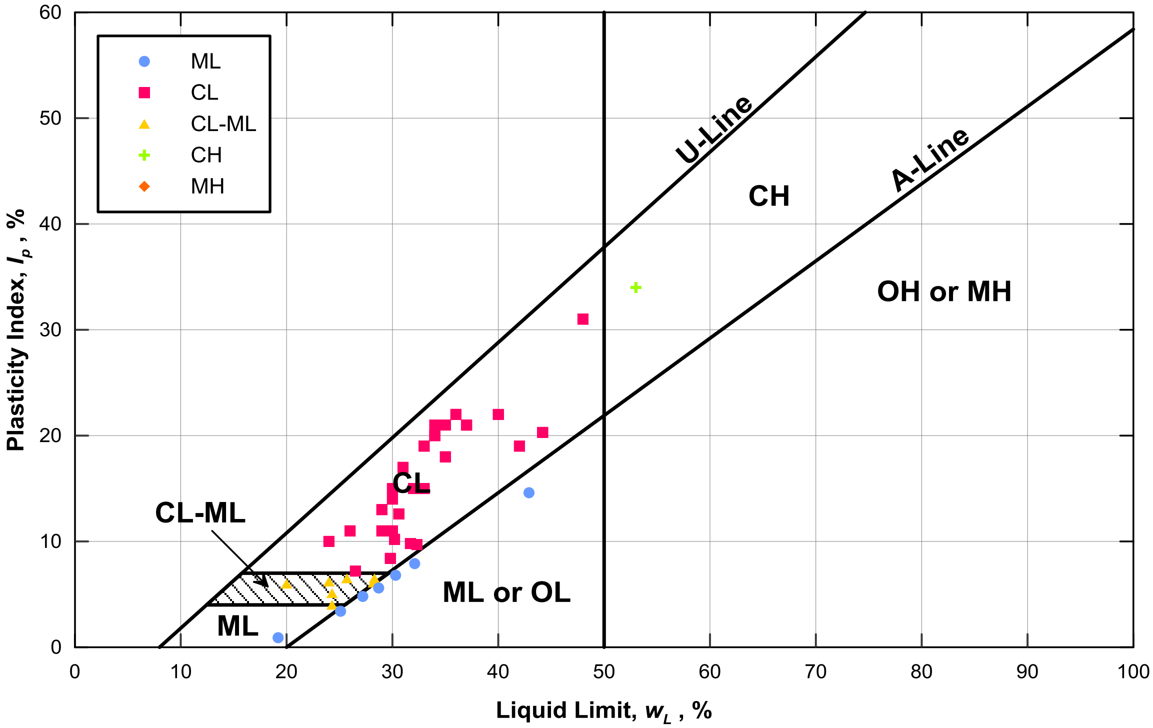


Figure 6.2-6
Plasticity Characteristics of Native Soils Encountered

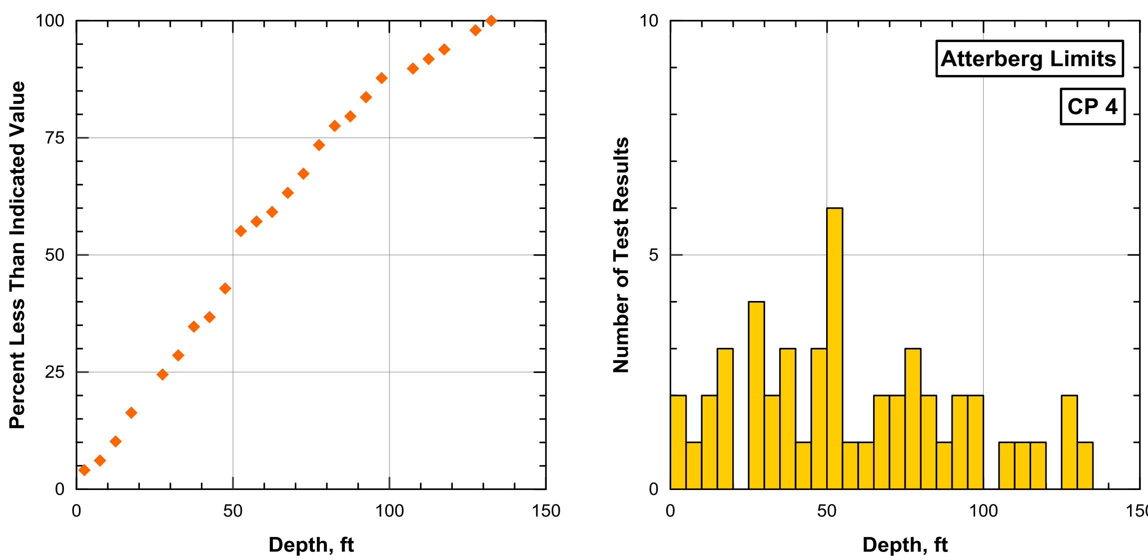


Figure 6.2-7
Probability and Frequency Distribution of Atterberg Limits Tests with Depth

For purposes of ground characterization, it is convenient to cluster soil types into coarse-grained and fine-grained categories that rely primarily on percent fines and expectations of engineering behavior. For example, fine-grained materials are categorized based on the potential to exhibit a discernible undrained response during transient loading. The definitions provided in Table 6.2-3 have been adopted to facilitate further data analysis and enable the baselining of engineering properties in the highly stratified geologic conditions encountered in the PE4P GI for CP4.

Table 6.2-3

Definition of Coarse-Grained and Fine-Grained Categories for Native Soils

Category	Source of Material Identification	Material Types
Coarse-Grained	Borehole Samples by USCS	GP, SW, SM, SP-SM, SP, SW-SM SM/ML, SC, SC-CL
	CPT data by SBT_N [a]	5, 6, 7, and 8
Fine-Grained	Borehole Samples by USCS	ML, CL, MH, CL-ML, CL/SC, CL-ML/SM, ML/SM
	CPT data by SBT_N [a]	1, 2, 3, 4, and 9

[a] SBT_N after Robertson 1990.

Spatial evaluations of interbedded coarse- and fine-grained deposits were undertaken for boreholes and CPTs and are presented in Figure 6.2-8 and Figure 6.2-9, respectively, for both above and below 20-foot depths. The separation of data above 20 feet is provided to allow closer assessment of shallow conditions along the alignments, which may be more relevant for particular structures and engineering applications.

The borehole results reflect intermittent sampling, typically at 5-foot intervals, and therefore provide less resolution with respect to stratigraphic changes in interbedded environments than CPT results. The CPT results yield soil classification indirectly through soil behavior type but provide near-continuous results with depth.

The upper chart of Figure 6.2-8 and Figure 6.2-9 shows the variability and randomness exploration-by-exploration of coarse- and fine-grained profiles in the top 20 feet bgs. The randomness is exacerbated by presenting the data over 20 feet in a profile that could easily vary between sandy silt and silty sand, as a single 5-foot sample interval of silt could skew the results. The randomness is an indication that the soils encountered during construction will vary from coarse (silty sands) to fine (sandy silts and sandy clays) over relatively short distances.

As described, the baseline geotechnical design parameters are presented in Section 6.6 and are divided by coarse- and fine-grained behavior. To characterize the interbedded stratigraphy of the CP4 soils encountered during the PE4P investigation as a practical, assumed design soil profile, a distribution of coarse- and fine-grained soils with depth will be required. The baseline distribution will not reflect the local variations captured by site-specific points of exploration. Baseline distribution of coarse- and fine-grained soils is intended to provide reasonable guidance for design soil types and corresponding baseline design values in an assumed soil profile applicable for bid design.

These baseline distributions of coarse- and fine-grained soils are presented in Table 6.2-4. Because the CPTs provide a near-continuous data profile, proposed baseline distributions of coarse- and fine-grained soils rely most heavily on the CPT results.

Table 6.2-4
Baseline Soil Type Distribution of Coarse- and Fine-Grained Soil

Starting	Ending	Soil Type Distribution (%)			
		Above 20 ft bgs		Below 20 ft bgs	
		Coarse	Fine	Coarse	Fine
Start of CP4	Approaching Woollomes Avenue	60	40	30	70
Woollomes Avenue	1,500 ft north of McCombs Avenue	60	40	50	50
1,500 ft north of McCombs Avenue	End of CP4	75	25	70	30

To implement these distributions, the design-builder should assume that the minor soil type is distributed evenly in approximately 5-foot-thick layers throughout the major soil type.

For example, for design of a 100-foot-long pile commencing below a 20-foot-deep pile cap in the northerly portion of CP4, 70 feet of the design profile should be assumed to use the fine-grained baseline engineering parameters and 30 feet of the design profile should be assumed to use the coarse-grained baseline engineering parameters. The 30 feet of coarse-grained soils should be distributed evenly as six 5-foot layers spaced throughout a fine-grained soil profile comprising six 11-foot-8-inch-thick fine-grained layers.

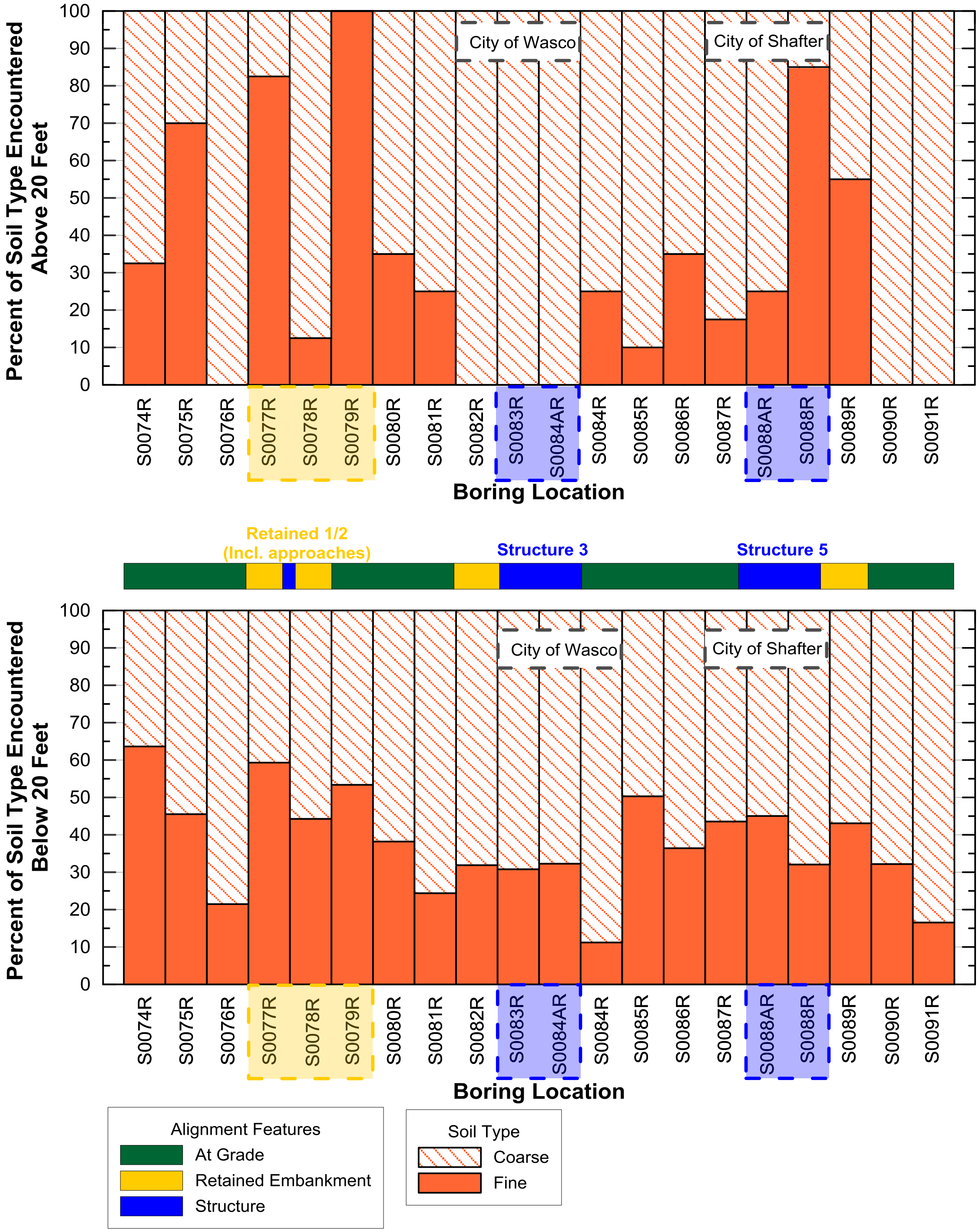


Figure 6.2-8
Percentage of Soil Type Above/Below 20 Feet bgs by Borehole Location

Note: Highlighted borehole (or CPT) labels distinguish test locations occurring within areas of major work. Yellow highlight corresponds to Retained Structure 1/2 and includes Structure 1 and approach embankments (comprising parts of At-Grades 2 and 3). Blue highlight corresponds to Structures 3 and 5 (elevated). Refer to Table 2.3-1 for descriptions of significant structures.

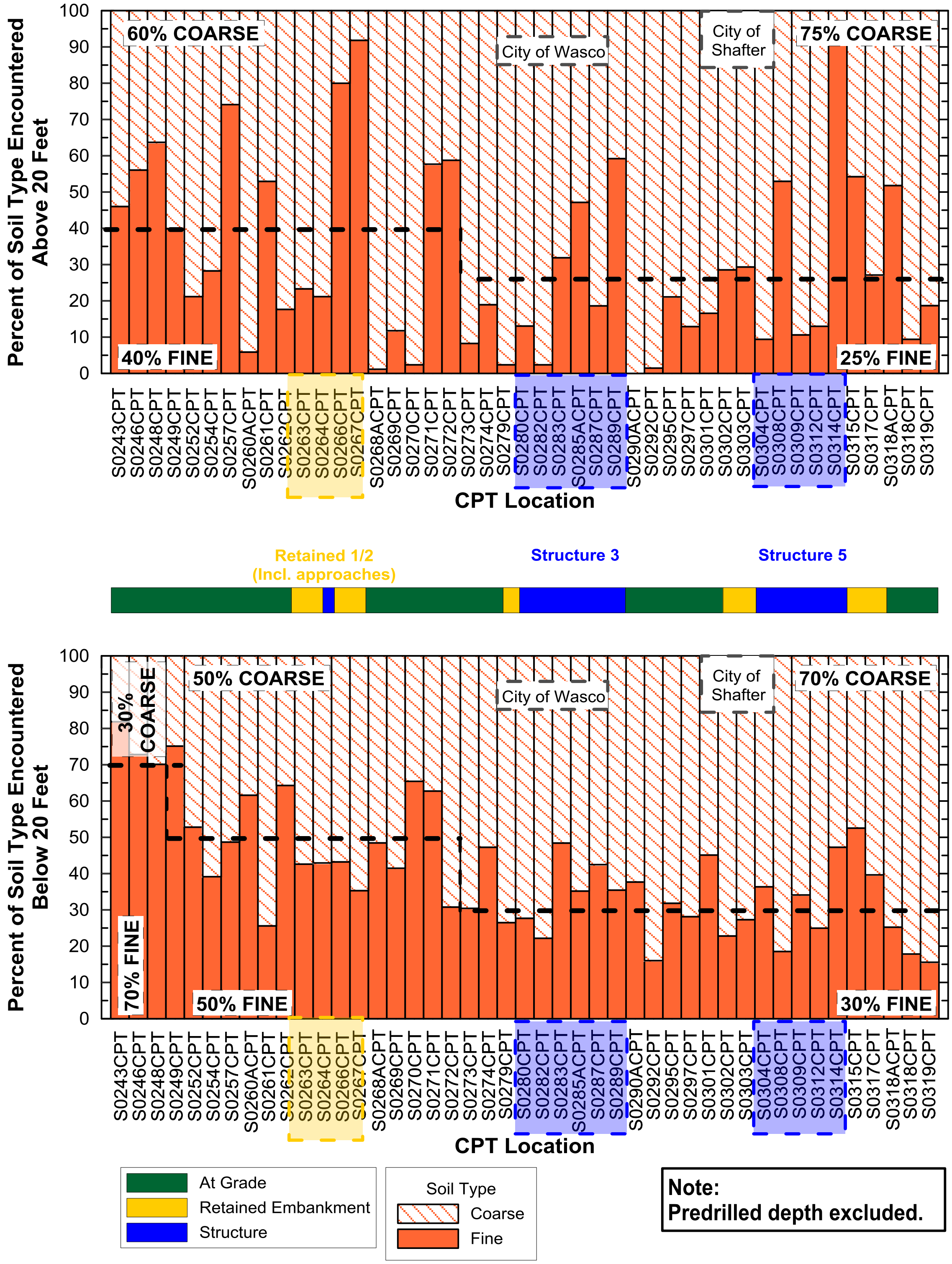


Figure 6.2-9
Percentage of Soil Type Above/Below 20 Feet bgs by CPT Location

Note: Highlighted borehole (or CPT) labels distinguish test locations occurring within areas of major work. Yellow highlight corresponds to Retained Structure 1/2 and includes Structure 1 and approach embankments (comprising parts of At-Grades 2 and 3). Blue highlight corresponds to Structures 3 and 5 (elevated). Refer to Table 2.3-1 for descriptions of significant structures.

6.3 Groundwater Level

Baseline design and construction groundwater levels are provided in Table 6.3-1. Design groundwater levels represent projected long-term levels for the design of permanent structures and allow for the potential reestablishment of historically high levels. Construction groundwater levels represent recent levels as observed during the PE4P GI.

Table 6.3-1
Baseline Groundwater Levels for Design and Construction

Starting	Ending	Design Groundwater Baseline Depth (ft)	Construction Groundwater Baseline Depth (ft)
Start of CP4	Approaching Woollomes Avenue	10	20
Woollomes Avenue	Approaching Taussig Avenue	50	75
Taussig Avenue	End of CP4	80	125

Shallower, perched groundwater will occur in the interbedded soils encountered along CP4. Open water retention/percolation ponds also exist along the alignment and in some cases lie directly within the proposed footprint of the alignment. Such conditions are discussed further in Section 8.0.

6.4 Contaminated Soil

The GI did not encounter or test for contaminated material; however, it may exist within the CP4 project area. No baseline for contamination is provided herein.

6.5 Corrosivity

6.5.1 Soil Chemistry

Corrosion tests were performed on four soil samples to evaluate the corrosion potential for buried iron, steel, mortar-coated steel, and reinforced concrete structures. Baseline values of soil corrosion parameters for existing fill and native soils are presented in Table 6.5-1.

Comparison with Caltrans criteria presented in Section 6.7.7 indicates that the soil samples tested did not exceed the criteria for chlorides, sulfates, minimum resistivity, and pH. Comparison with durability requirements of the American Concrete Institute (ACI 318-11) indicates that sulfate concentration range (<1,000 ppm) and chloride concentration range (<600 ppm) are unlikely to impact reinforced concrete structures.

Although the results of the samples tested do not indicate the presence of a corrosive environment, corrosive conditions may be present along the proposed HSR corridor. The GI completed by the design-build Contractor for final design should evaluate the corrosion potential (soil corrosion, stray current corrosion) at specific sites where the proposed improvements may be sensitive to a corrosive environment.

For bidding purposes, assume that a corrosive environment exceeding the Caltrans criteria (Section 6.7.7) but no more severe than "moderate" corrosivity by ACI standards, is present over 10% of the project area. When costing, this allocation should be distributed evenly across all structures.

Table 6.5-1
Baseline Corrosion Parameters

Test	Test Reference	No. of Tests[a]	Range of Values	Mean Value	Standard Deviation	Baseline Value
Minimum Resistivity (ohm-cm)	ASTM G 57	5	1651 to 5112	3067	1460	**1650**
pH	ASTM D 4327	5	7.05 to 8.6	8	1	**7**
Sulfate (ppm)	ASTM D 4327	4	50 to 600	278	251	**600**
Chloride (ppm)	ASTM D4327	4	28 to 47	37	10	**50**

[a] All tests were conducted on bulk samples comprising soil from within the first 5 feet of depth.

6.5.2 Groundwater Chemistry

Groundwater corrosivity parameters are based on the results of one sample collected from the piezometers at S0078R located adjacent to Poso Creek, presented below in Table 6.5-2.

Table 6.5-2
Groundwater Chemistry Test Results

Test	Test Reference	Borehole ID			
		S0077R	S0078R	S0083R	S0088AR
pH	SM 4500-H^+B		9.6		
Calcium (mg/L)	EPA 200.7		5.6		
Bicarbonate Alkalinity as $CaCO_3$ (mg/L)	SM 2320B		72		
Specific Conductance (umhos/cm)	SM 2510B	Dry	284	Dry	Dry
Total Dissolved Solids (mg/L)	SM 2320B		177		
Chloride (mg/L)	EPA 300.0		20.6		
Sulfate as SO_4 (mg/L)	EPA 300.0		25.3		

6.6 Engineering Parameters of the Subsurface Materials

6.6.1 Existing Fill and Near-Surface Soils

The primary purpose for providing baseline parameters for existing fill and near-surface soil is to facilitate the design of temporary and permanent shallow foundations, pavements, and earthworks. Refer to Section 6.2.1 for description of near-surface soil types encountered in the GI, comprising (potential) existing fill and near-surface soils.

No undisturbed laboratory tests were performed on existing fill and near-surface soils because the bulk samples collected were highly disturbed and were taken from drilling cuttings. Laboratory tests performed included Modified Proctor Compaction, California Bearing Ratio, resistance value (R-value), moisture content, and fines content.

In situ properties of potential existing fill and near-surface soils, including total unit weight and natural water content, are presented in Table 6.6-1 and are based on too few tests to develop statistically significant conclusions appropriate for large-scale earthworks. Despite the limited data, the total unit weight and natural water content test results provide a range of values generally within expectation for the soil types encountered, and baseline values representative of the anticipated conditions have been chosen from the range.

Table 6.6-1
Baseline In Situ Properties for Potential Existing Fill and Near-Surface Soils

		Total Unit Weight[a], pcf (γ_t)	Natural Water Content, % (w_c)
COARSE	No. of Tests	5	13
	Range	110.7 to 138.3	2.8 to 11.5
	Assumed Baseline	**120**	**8.6**
FINE	No. of Tests	3	6
	Range	106.6 to 125.6	9.9 to 18.2
	Assumed Baseline	**117**	**14.0**

[a] Based on lined modified California sampler data

Moisture content will vary significantly by season with the quantity and timing of rainfall. The as-measured, in situ moisture contents for coarse-grained materials approximate the optimum moisture content from compaction tests Table 6.6-2. The approximation is coincidental and should not lead the bidder to assume that no moisture conditioning of site soils will be required. Moisture conditioning assumptions are described further in Section 6.6.1.

Baseline compaction parameters of potential existing fill and near-surface soils are provided in Table 6.6-2. The number of tests performed was limited, and generally insufficient to support results that may differ from engineering expectations. Baseline values have been chosen based on engineering judgment regarding expected material properties to be encountered in the near surface based on the GI.

Baseline in situ strength parameters for existing fill and near-surface soils are provided in Table 6.6-3. Laboratory strength testing consisted of two direct shear tests on existing fill samples below 10 feet depth, resulting in friction angles of 28° and 36°. Fill and near-surface material vary widely, and the baseline strength parameters provided below rely on engineering judgment for similar materials based on perceived composition and in situ density/consistency.

Bulking/swell factors used to estimate earthwork volumes typically range between 10% for sand and gravel to about 30% for clay. Shrinkage factors range from about 10% for sand to about 30% for clay. For bidding purposes, assume potential existing fill and near-surface soils have bulking/swell factors of 20% and a shrinkage factor of 10%.

Table 6.6-2
Baseline Earthworks Parameters for Existing Fill and Near-Surface Soils

		Fines Content (%)	Maximum Dry Density ($\gamma_{d,max}$)	Optimum Moisture Content (w_o)	California Bearing Ratio (%)	R-Value[a]
COARSE	**No. of Tests**	13	16	16	4	15
	Range	17.4 to 54.4	119.3 to 133.4	6.5 to 11.2	6 to 33	10 to 66
	Assumed Baseline	**30**	**128**	**9.1**	**8**	**20**
FINE	**No. of Tests**	–	4	4	2	3
	Range	–	115.3 to 127.7	8.9 to 14.3	13.5 to 55	5 to 21
	Assumed Baseline	–	**120**	**11.4**	**3**	**5**

[a] One treated R-Value of 84 is not reported in this table. R-Values noted above are for untreated samples.

Notes:

Soils tested comprise hand auger samples collected over depths of 0 feet to 5 feet. Borings were undertaken adjacent to existing roads, and locations may reside outside of the footprint of the proposed alignments. Conditions elsewhere (e.g., nearby agricultural land) may vary.

Table 6.6-3
Baseline In Situ Strength Parameters for Potential Existing Fill and Near-Surface Soils

Soil Type	Effective Strength Parameters [a]		Undrained Shear Strength [a], s_u (psf)
	Friction Angle, Φ′, (°)	Cohesion (psf)	
Coarse	29	- [b]	N/A
Fine	28	- [b]	1,000

[a] Laboratory testing to assess strength of existing fill and near-surface soil was not undertaken. Values presented are based on engineering judgment for typical values based on material type and perceived in situ density or consistency. Strength of reworked material will vary by compactive effort and moisture conditions.

[b] Effective cohesion shall be taken are zero, except for purposes of earthworks slope stability assessment, where a value of 50psf shall be adopted.

6.6.2 Native Soils

The baseline description of native soils encountered in the CP4 alignments is provided in Section 6.2.2 and indicates predominantly interbedded coarse- and fine-grained soils. This section pertains to native soils below existing fill or near-surface soils as defined in Section 6.6.1.

A single set of engineering parameters for the native soils are provided as a baseline for design across the entire contract package. While it is likely that conditions will deviate along the alignment, exploration spacing is too wide to develop stratigraphic trends and substantiate distinctions of significant engineering consequence targeting specific areas or structures.

Table 6.6-4
Baseline Engineering Properties for Native Soil

Material	Depth Regime	Value	Total Unit Weight γ_t (pcf)	Soil Modulus[a] E_s (tsf)	Corrected Blow Count SPT N_{60} (bpf)	CPT Tip Resistance q_c (tsf)	Effective Friction Angle Φ' (deg)	Effective Cohesion Intercept[b] c' (psf)	Undrained Shear Strength[c] s_u (psf)	Shear Wave Velocity V_s (ft/sec)
Coarse-grained soils	Above 20 feet	Range	106 to >137	35 to 1,679	3 to 72	8.7 to 420	21 to 37	0 to >1,000	–	**5.5D+ 875 ≤2,500**
		Baseline	**120**	**300**	**13**	**60**	**32**	**50**	**–**	
	Below 20 feet	Range	95 to >137	107 to >2,000	10 to >99	27 to 771	23 to 42	0 to >1,000	–	
		Baseline	**125**	**300+9*(D-20) ≤1,200**	**13+0.44(D-20) ≤75**	**60+3.7*(D-20) ≤400**	**36**	**50**	**–**	
Fine-grained soils	Above 20 feet	Range	107 to >130	65 to >2,000	3 to 71	3.9 to 233	-	-	1,097 to >5,000	
		Baseline	**118**	**300**	**18**	**25**	**30**	**100**	**2,400**	
	Below 20 feet	Range	95 to >137	88 to >2,000	4 to >99	7.1 to 476	23 to 38	0 to >1,000	1,010 to >5,000	
		Baseline	**125**	**300+5.5*(D-20) ≤1,500**	**18+0.23(D-20) ≤50**	**25+(D–20)/3 ≤100**	**32**	**200**	**2,400+30*(D-20) ≤6,000**	

[a] Range of soil modulus relies upon correlation with CPT data; refer to Appendix A.

[b] For coarse-grained soil, effective cohesion is often an apparent cohesion used only to adjust for the non-linearity at low effective stresses typical of simplified Mohr-Coulomb failure criteria. For baseline purposes, effective cohesion shall be ignored for all failure surfaces through disturbed soil, or along undisturbed native soils where confined by less than 10 ft of overburden.

[c] Ranges given are based on laboratory TXUU data.

Where a baseline value is provided as an equation, 'D' represents depth bgs in feet.

6.6.2.1 Standard Penetration Test Blow Count

SPT blow counts were recorded during soil sampling in boreholes and corrected to SPT N_{60} values using the results of hammer efficiency measurements recorded during the site exploration. For comparison, CPT tip resistance data were correlated to equivalent SPT N_{60} values as described in Appendix A. Histograms and statistical data of SPT N_{60} are presented in Appendix A. Histogram plots were capped at a maximum value of 100 blows per foot.

The baseline SPT N_{60} blow counts for coarse- and fine-grained soils above and below 20 feet bgs are provided in Table 6.6-4. Figure 6.6-1 illustrates N_{60} and baseline values for coarse- and fine-grained material.

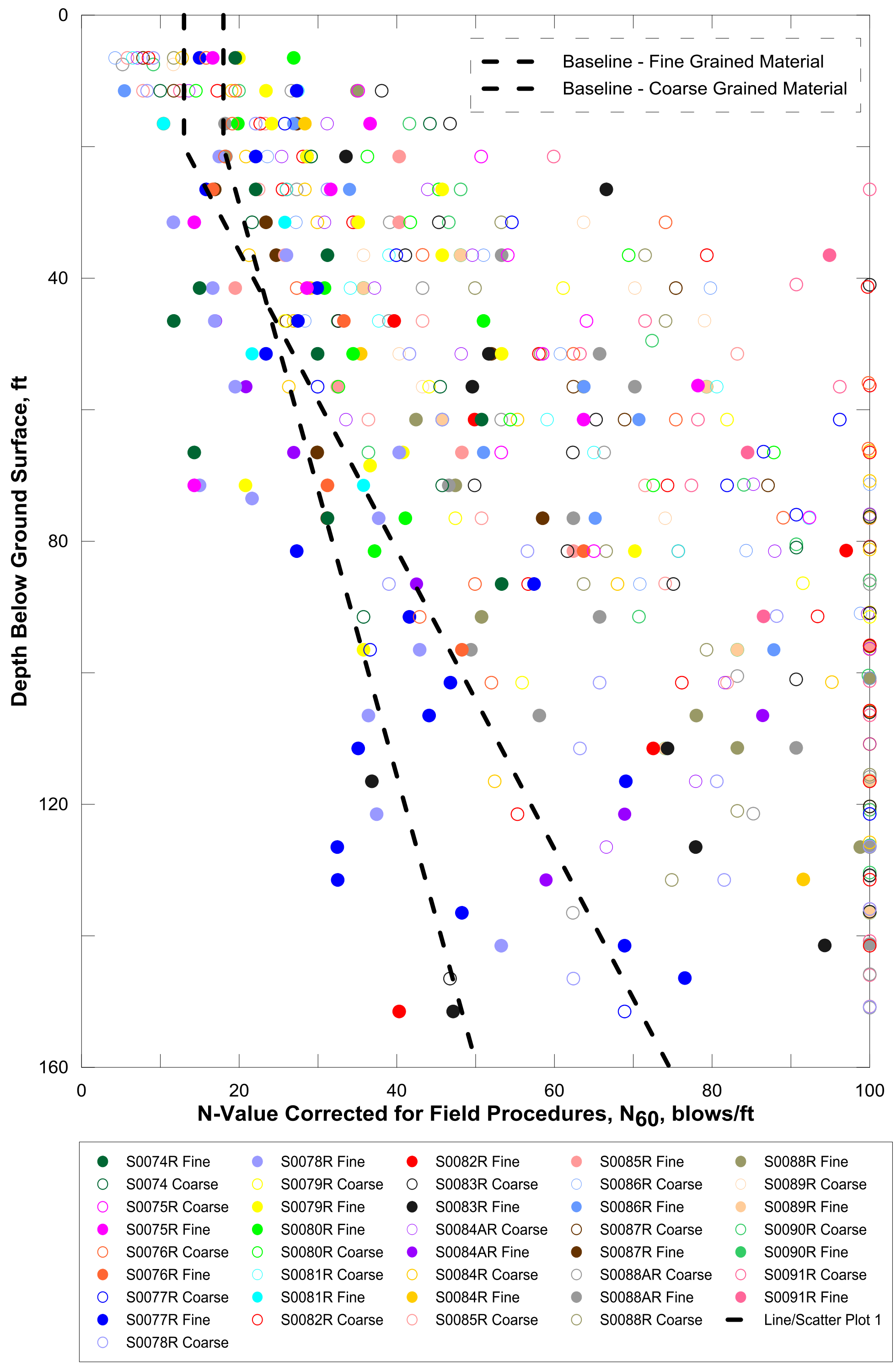


Figure 6.6-1
SPT N_{60} Results

Hardpan soils were not encountered during the CP4 GI based on site observations during the GI. Resistance at deep levels may be associated with varying degrees of potential cementation, as was experienced in the several boreholes at depths typically below 40 feet.

6.6.2.2 Cone Penetration Test Tip Resistance

The baseline CPT cone resistance (q_c) for coarse- and fine-grained soils above and below 20 feet bgs are provided in Table 6.6-4.

CPT cone resistance data are summarized as histograms, including mean, median, and standard deviation results, in Appendix A.

6.6.2.3 Unit Weight

The total unit weight baseline values are provided in Table 6.6-4.

Histograms of CPT correlated unit weight and densities from drilling samples are presented in Appendix A.

6.6.2.4 Undrained Shear Strength

Undrained shear strength is a design parameter relevant to fine-grained materials. Fine-grained materials can exhibit cohesive behavior that retards the drainage of pore water in saturated or partially saturated soil. This results in an "undrained" response to applied load, whereby excess pore water pressure is generated, and the initial resistance provided by the soil is represented by an undrained shear strength.

The usage of drained (effective) or undrained strength parameters is application-specific and to be determined by the design builder.

Undrained shear strength for fine-grained native soil of CP4 was determined from triaxial unconsolidated undrained (TXUU) shear strength tests on 6 borehole samples taken above 20 feet bgs, and 20 tests on samples from below 20 feet bgs. Undrained shear strength was also estimated by correlation with CPT q_c data. The details for this data are presented in Appendix A.

Overall, the CPT data indicate a higher undrained shear strength than suggested by the TXUU tests results, particularly below 20 feet. This is not uncommon, as laboratory samples are subject to disturbance and relaxation during transport, extrusion, and testing. By comparison, penetration response during CPT testing can be more representative of undisturbed in situ conditions. Undrained shear strength for fine-grained native soil in CP4, as estimated from TXUU and CPT data, is presented in Figure 6.6-2. For reference, a SHANSEP line depicting shear strength with depth for an assumed OCR = 1.5 is provided, which represents theoretical shear strength of slightly overconsolidated consolidated soils. The relationship of the test results and correlation data to the SHANSEP line suggest that the materials encountered on-site are generally overconsolidated.

The baseline undrained shear strengths for fine-grained Native Soil above and below 20 feet bgs is presented in Table 6.6-4.

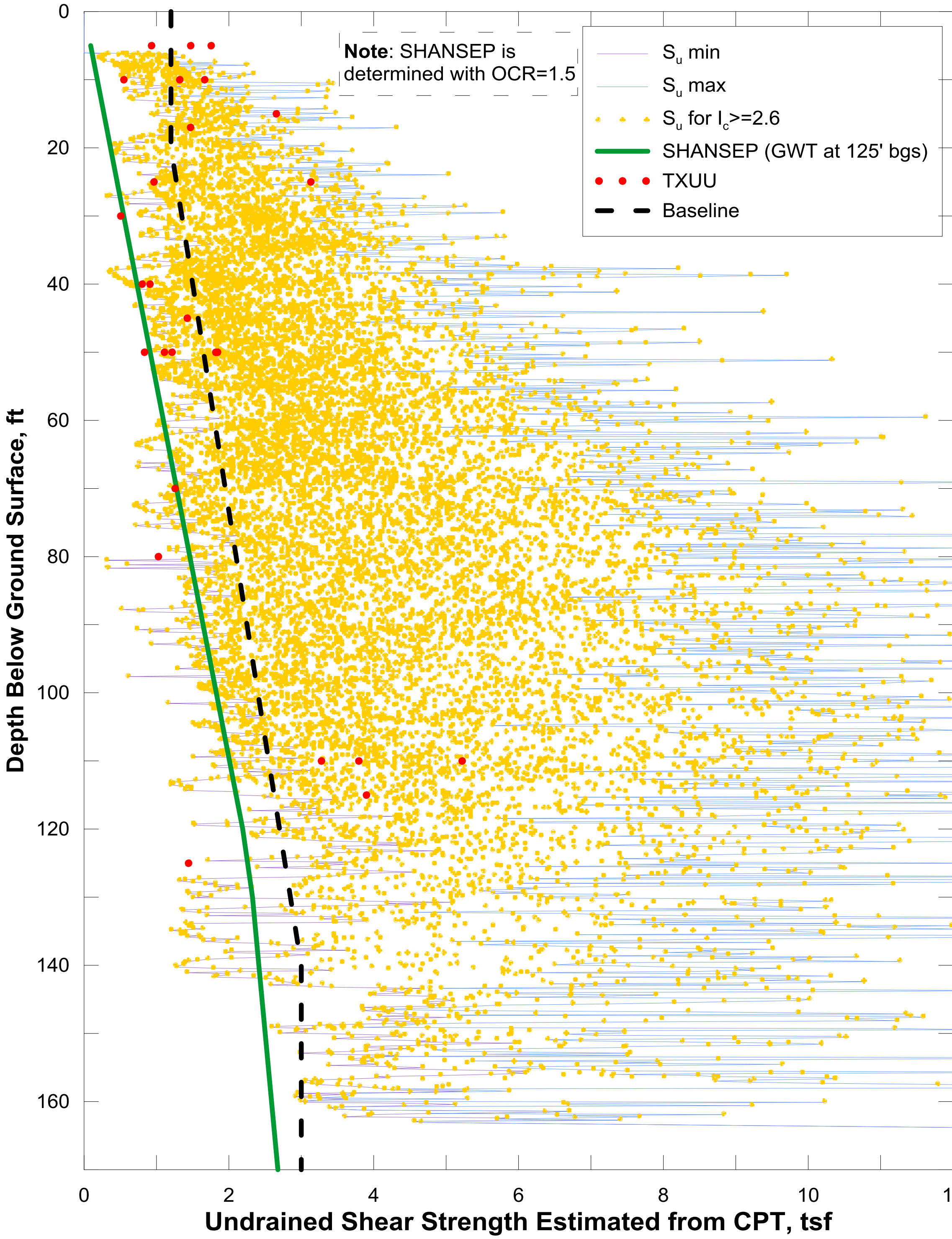


Figure 6.6-2
Undrained Shear Strength of Fine-Grained Material in CP4 Based on CPT and TXUU Results

6.6.2.5 Effective Shear Strength

Baseline effective stress strength parameters (Φ′ and c′) for coarse- and fine-grained soils above and below 20 feet bgs are provided in Table 6.6-4.

Effective shear strength parameters for CP4 include effective friction angle (Φ′) and effective cohesion (c′). The effective friction angle for the predominantly coarse-grained soil of CP4 was estimated from CPT and SPT blow count correlations. Laboratory testing was performed to evaluate effective stress parameters, and comprised triaxial consolidated drained tests (TXCD) and direct shear (DS) tests on driven samples from California Modified and piston samplers.

Direct shear tests were undertaken on 53 soil samples, and the results are presented graphically in Figure 6.6-3. The distribution of direct shear tests included 8 fine-grained soil samples and 45 coarse-grained soil samples. For reference, the baseline parameters of Table 6.6-4 are illustrated on Figure 6.6-3.

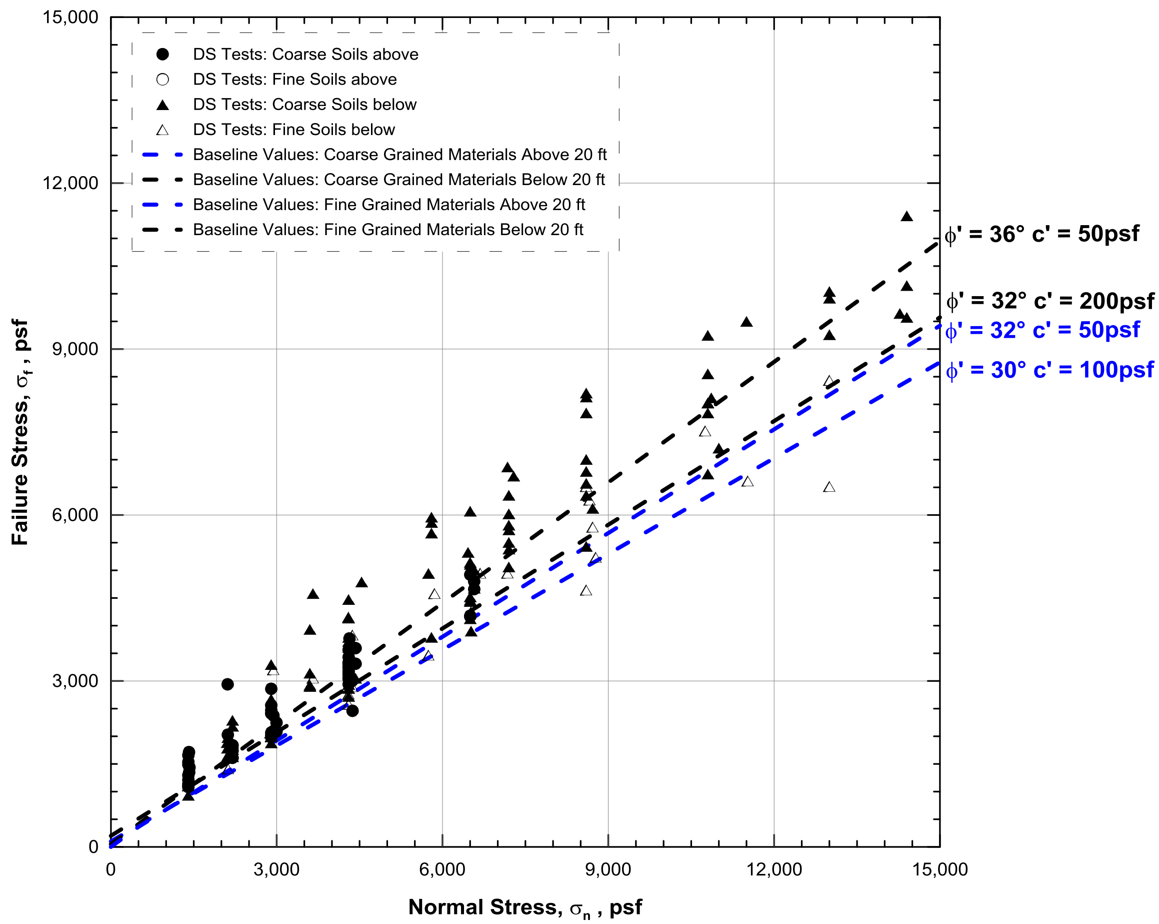


Figure 6.6-3
Results of Direct Shear Tests for Native Soil Samples Obtained in CP4

Consolidated drained triaxial tests were undertaken on 2 soil samples from above 20 feet bgs and 16 soil samples from below 20 feet bgs, and the results are presented in Appendix A. The distribution of tests includes 11 fine-grained soil samples and 7 coarse-grained soil samples. The results suggest effective friction angle trends of approximately 31° and 29° for fine-grained soils

above and below 20 feet, respectively, and 36° for coarse-grained soils below 20 feet. No tests were performed on coarse soils obtained above 20 feet bgs.

The statistical results of the CPT and SPT correlations and laboratory test data are presented in Appendix A.

6.6.2.6 Soil Modulus

Baseline values of soil modulus (E_s) for coarse- and fine-grained soil above and below 20 feet bgs are presented in Table 6.6-4.

Typical values for soil modulus as presented by American Association of State Highway Transportation Officials (AASHTO, 2010) are presented in Table 6.6-5.

Table 6.6-5
Published Soil Modulus (AASHTO 2010)

Soil Modulus, E_s (tsf)	
Clay	
Soft Sensitive	25 to 150
Medium Stiff to Stiff	150 to 500
Very Stiff	500 to 1000
Silt	
20 to 200	
Sand	
Loose	100 to 300
Medium Dense	300 to 500
Dense	500 to 800

Soil modulus of course-grained soil was estimated using correlations with $N_{1(60)}$ and CPT q_c, as detailed in Appendix A. The SPT correlation resulted in lower estimates of soil modulus than CPT-based correlations.

The baseline for coarse soils above 20 feet bgs is representative of loose to medium-dense sand, and is taken to increase linearly with depth thereafter from medium dense near 20 feet bgs to very dense below 75 feet bgs. The baseline is illustrated against the CPT correlation for soil modulus of coarse material in Figure 6.6-4.

The baseline for fine-grained soils relies upon correlation with s_u from TXUU and CPT q_c, as described in Appendix A. This yields estimates of soil modulus that are proportional to the s_u values of Figure 6.6-2. Baselines are provided in Table 6.6-4 and are representative of medium stiff to stiff cohesive soil above 20 feet bgs, and of an increase from stiff cohesive soil near 20 feet bgs to very stiff cohesive soil below approximately 50 feet bgs.

Histograms and other statistical data used to determine soil modulus from SPT and CPT correlations are presented in Appendix A.

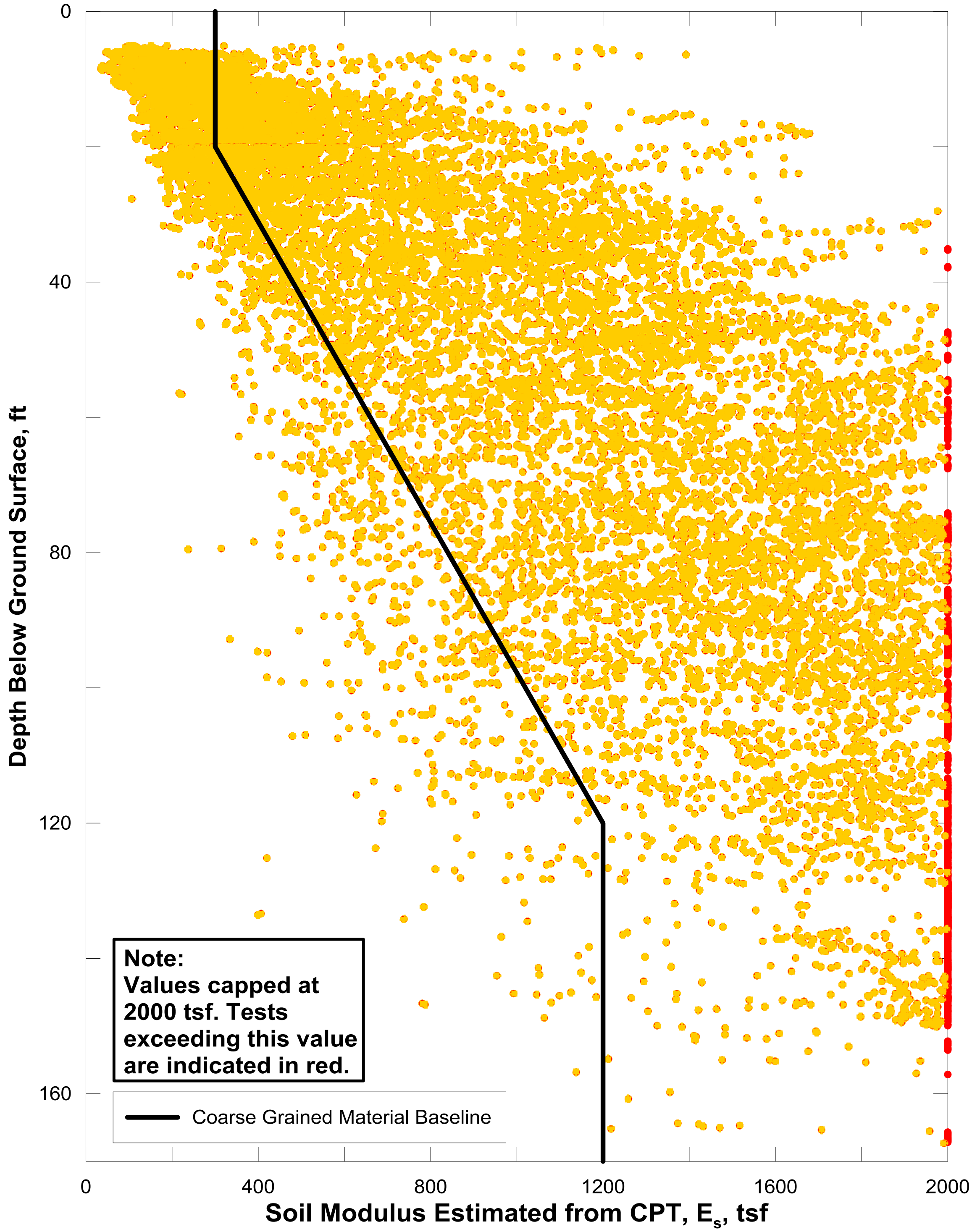


Figure 6.6-4
Soil Modulus of Coarse Soil from Correlation with CPT Data

6.6.2.7 Shear Wave Velocity

Shear wave velocities averaged over the upper 100 feet (~30 meters) of soil, V_{s30}, are presented in the GDR. Baseline V_s values based on the available data are provided in Table 6.6-4 and presented in Figure 6.6-5.

Shear wave velocity measurements were taken in CPTs S0266CPT, S0279CPT, S0289CPT, S0297CPT, S0312CPT, S0318CPT and via PS logging in boreholes S0077R, S0082R, S0088R, and S0091R. The seismic Site Class boundary between Class C and Class D soil is shown for reference only.

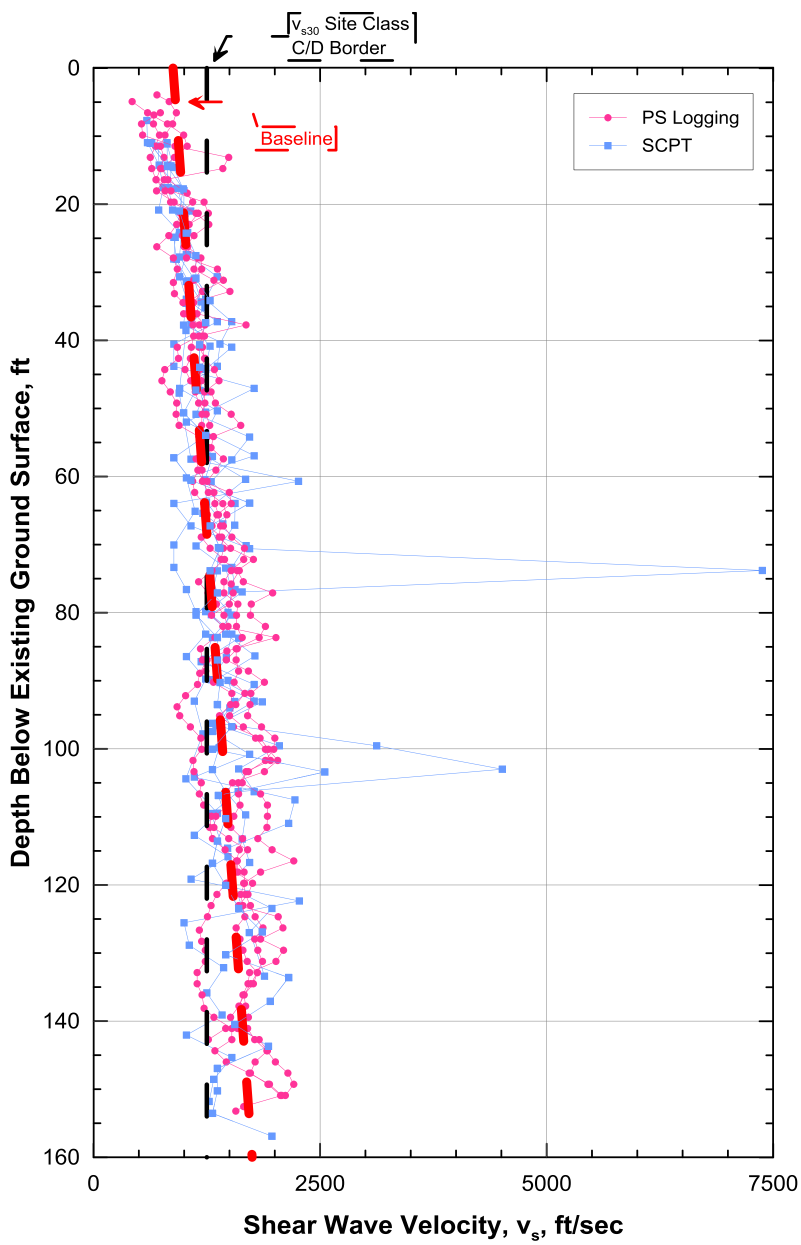


Figure 6.6-5
Shear Wave Velocity Measurements and Baseline Value

6.6.2.8 Modulus of Vertical Subgrade Reaction

Figure 6.6-6 shows the range of Modulus of Vertical Subgrade Reaction (k'_v) applicable for sands, based on the results of plate load tests on a 1-foot by 1-foot plate. The baseline subgrade modulus is determined from the baseline SPT N_{60} blow count correlated to the typical vertical subgrade reaction modulus values shown in Figure 6.6-6. Modulus values should be adjusted for the width of the foundation element. A bi-linear relationship between subgrade modulus and relative density was utilized.

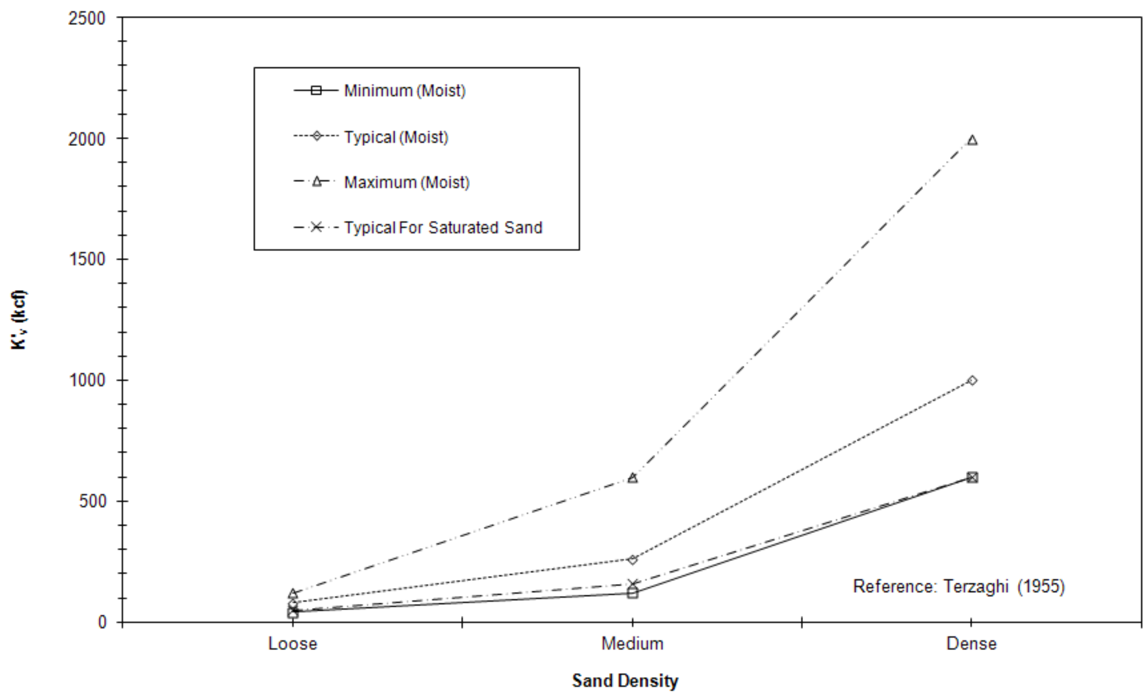


Figure 6.6-6
Modulus of Vertical Subgrade Reaction (Terzaghi 1955)

6.6.2.9 L-Pile Parameter: Soil Modulus, K

Typical values of Soil Modulus (K), or Modulus of Horizontal Subgrade Reaction (k_h), for granular soil range from 20 to 225 pounds per cubic inch (pci) based on an assessment of the relative density of the sand and the effect of a submerged or dry condition (FHWA-NHI-10-16). Typical values of k_h published by the American Petroleum Institute (API 1987) are shown on Table 6.6-6.

Table 6.6-6
Typical Static Modulus of Horizontal Subgrade Reaction, k_h (API 1987)

	Subgrade Reaction k_h by Relative Density (pci)		
	Loose	**Medium Dense**	**Dense**
Sand Below Water Table	20	60	125
Sand Above Water Table	25	90	225

For bidding purposes, assume K varies above and below 20 feet bgs. For granular soils above 20 feet bgs, a baseline value of 40 pci for static conditions and 20 pci for cyclic loading may be adopted. For granular soils below 20 feet bgs, a baseline value of 80 pci for static conditions and 40 pci for cyclic conditions may be adopted.

For fine-grained native soil, assume K equal to 500 pci above 20 feet bgs, and 1,000 pci below 20 feet bgs for static loading. For dynamic loading, assume K equal to 200 pci above 20 feet bgs, and 400 pci below 20 feet bgs.

These values do not assume liquefied soil conditions.

Usage of these parameters is intended for and limited only to lateral pile analysis using L-Pile software.

6.7 Baseline Soil Behavior

6.7.1 Earthworks Near-Surface

For bidding purposes, assume near-surface soil is loose to medium dense and soft to stiff and can be excavated with conventional grading equipment such as dozers, scrapers, and track mounted excavators. Where excavated vertically, existing fill will not remain stable. Excavations in existing fill will be prone to raveling within a few minutes where it is dry, and will flow where it is wet. It is anticipated that sloped cuts or temporary shoring will be required to maintain stability of excavation in existing fill.

Existing fill will require moisture conditioning prior to reuse and recompaction to achieve desired density. This will require adding water to soil that is dry of the optimum moisture content and air-drying soil that is wet of the optimum moisture content. Air drying during periods of rain (November through March) is assumed to be impractical. Stabilization through addition of lime may be applicable in some areas, where fine-grained soils are sufficiently clayey.

In general, cement or lime treatment may be feasible in cases where clayey soils are encountered in earthworks, but for bidding purposes assume it will not be necessary.

6.7.2 Cementation (Rippability)

The majority of soil in CP4 exhibits no cementation. Of the soil that does exhibit cementation, the majority is weak cementation with occurrences ranging to strong cementation, based on Table 6.7-1 (*Soil and Rock Logging, Classification, and Presentation Manual,* Caltrans 2010).

For bidding purposes, assume that existing fill and native soil in CP4 exhibit weak cementation at most.

Table 6.7-1
Cementation Criteria (Caltrans 2010)

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

6.7.3 Stability

Excavations in native soils above groundwater may remain stable for sufficient time to allow for temporary shoring installation. This should be evaluated further by the Design Builder.

Native soils below the groundwater, or above the phreatic surface but subject to locally higher (perched) water, will experience sloughing or running conditions. Shallow areas may thus require benching or battering to provide stable conditions. Where deep foundations extend below the groundwater level for construction, temporary casing and/or drilling slurry will be required.

6.7.4 Shrink/Swell Potential

Native soil in CP4 is predominately coarse-grained and will not generally be subject to significant shrinking or swelling. Results of Atterberg limits tests suggest a low degree of shrink and swell potential (less than 18%) based on plasticity indices (Holtz 1959 and USBR 1974).

The majority of fine-grained native soil tested in the CP4 GI exhibited low to intermediate plasticity, with liquid limits between 25 and 35%. Only a single sample indicated high-plasticity clay. It is expected that fine-grained soils in CP4 will vary from a low to medium degree of shrink and swell potential.

As a baseline, the design-builder should assume that 50% of fine-grained soil encountered in CP4 exhibits low potential for shrink swell and that 50% exhibits a medium potential for shrink swell behavior. This distribution should be applied equally over all major structures.

6.7.5 Collapse and Expansion

Collapse and expansion potential has been investigated using the results of the GI and a method proposed by Mitchell and Gardner 1975, and Gibbs 1969. The results are presented in Figure 6.7-1.

In general, neither collapse nor expansion are expected to present significant construction issues on CP4, and no baseline is provided.

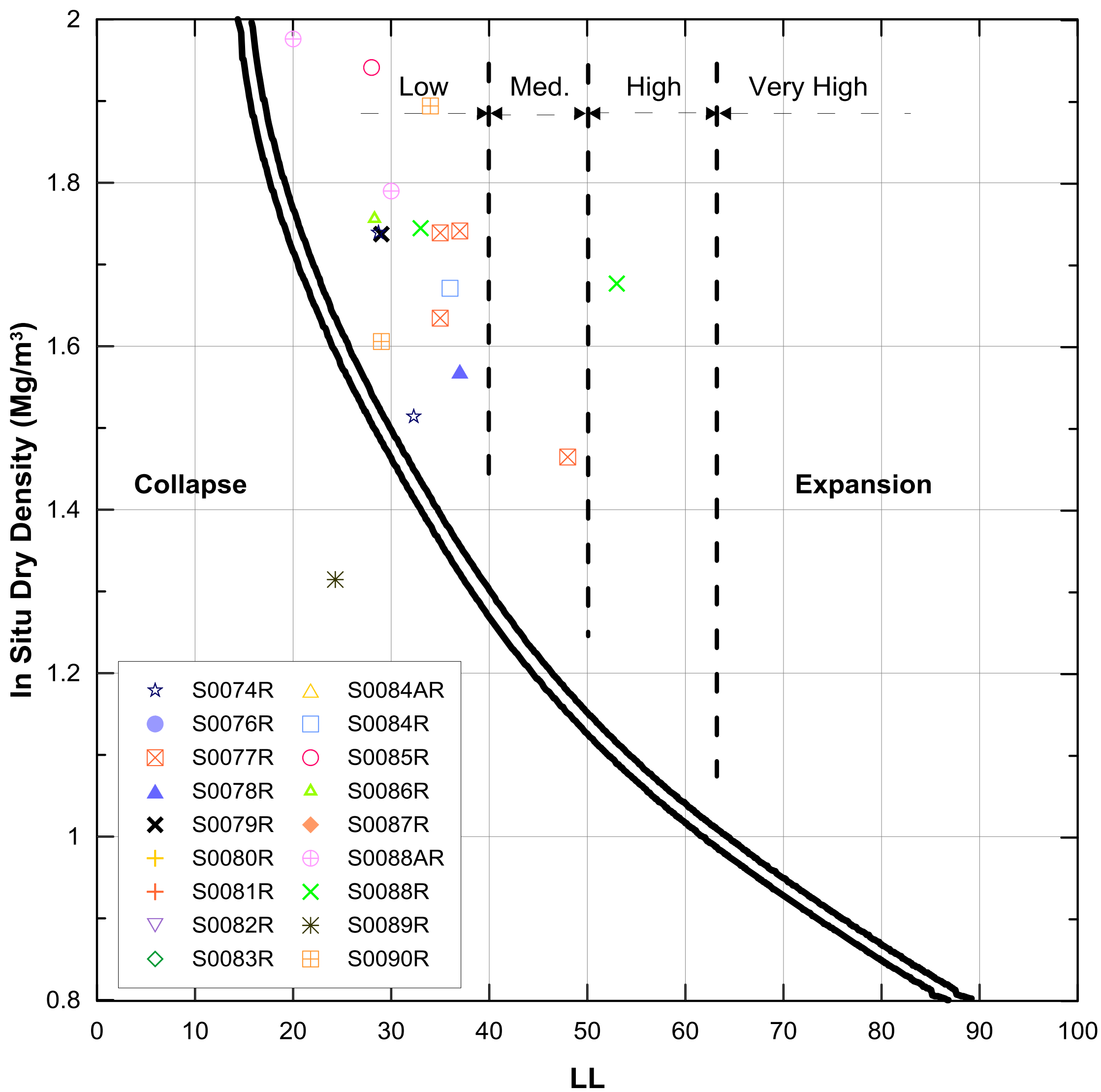


Figure 6.7-1
Collapsibility, Compressibility, and Expansion for Samples with both Liquid Limit and Dry Density

6.7.6 Land Subsidence

Refer to Section 4.4.4 and the GSHR and GDR for background on potential land subsidence issues along the project alignment. Unless directed otherwise by the Scope of Work, for bidding purposes assume that subsidence from groundwater pumping is not an impact to the project area.

6.7.7 Corrosion

For buried concrete and steel elements, Caltrans Corrosion Guidelines (2012) consider a site to be corrosive and/or require further testing if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Resistivity is 1,000 ohm-cm or less.
- Chloride concentration is 500 parts per million or greater.
- Sulfate concentration is 2,000 parts per million or greater.
- pH is 5.5 or less.

Further criteria relevant to corrosion in structural design may be found in the California Building Code and publications by the American Concrete Institute, the American Institute of Steel Construction, and others.

Baseline assumptions for bidding purposes have been provided in Section 6.5.1.

6.7.8 Long-Term Settlements

Any existing fill is expected to be either replaced or recompacted in areas of earthworks and properly compacted suitable material will not contribute to long-term settlements.

The native soils in CP4 comprise interbedded coarse- and fine-grained material. The fine-grained material is generally stiff to hard, and sufficiently overconsolidated to respond elastically and immediately to the application of new load from embankments. Where fine-grained soils may be present in a less stiff condition and shallow enough that embankment loads instigate consolidation settlement, it is expected that the drainage paths to coarse layers and the probable construction duration will be sufficient to build out the majority of settlement prior to placement of permanent track works.

Creep, or secondary settlement, is considered to occur following consolidation of fine-grained material, but can also occur in some coarser-grained materials. In general, secondary settlement associated with the overconsolidated native material underlying embankments is expected to be minor.

For bidding purposes, assume that long-term settlement associated with consolidation and creep in the native soil will not exceed the project design criteria for settlement after construction of permanent way tracks. This baseline presumes duration of up to two years between construction of earth embankments and the initial placement of permanent rail track.

It must be noted that additional long-term settlements may occur due to ground subsidence associated with continuation of groundwater extraction programs, as discussed further in the Section 6.7.6.

Section 7.0

Design Considerations

7.0 Design Considerations

7.1 Deep Foundations

Cast-in-drilled-hole (CIDH) piles are planned for the support of most railway structures. Driven piles are planned for most roadway overcrossing bridge abutments. Refer to the PE4P drawings for foundation types at specific locations. In general, the subsurface conditions have been baselined above and below a depth of 20 feet. Foundation designs should consider these baselines.

7.1.1 Cast-in-Drilled-Hole Piles

The preliminary design includes deep foundations consisting of cast-in-drilled-hole (CIDH) mono-piles and pile groups to support elevated structures and some roadway overcrossings. The selection of CIDH piles was driven by large foundation loads and stringent deflection criteria. Right-of-way constraints and proximity of existing surface structures influenced the preliminary pile type and size selection to those with manageable pile cap footprints.

7.1.2 Axial and Lateral Resistance

Axial resistances of CIDH piles are predominantly determined based on SPT N_{60} values, cone tip resistances, and laboratory undrained shear strengths. Baseline values recommended in Section 6.6 allow for estimating nominal skin friction, end-bearing resistance, and p-y curves. Nominal resistances should be determined in accordance with Caltrans amendments to AASHTO requirements as per the project Contract Documents.

Significant consideration in the design of deep foundations must be given to lateral load resistance. This resistance is likely to be limited by the stringent deflection criteria necessary to maintain the track-structure interaction criteria. Typical spans and long-span elevated structures will exert large lateral demands on foundations, potentially requiring additional piles for lateral resistance, enlarged pile caps, or post-tensioned CIDH piles.

7.1.3 Groundwater

Design of CIDH piles must consider the long-term possibility of groundwater fluctuations. The baseline design groundwater table depth for design of deep foundations ranges between 10 and 80 feet, as shown on Table 6.3-1. Perched water may exist at higher elevations.

7.1.4 Downdrag and Uplift Loads

Settlement adjacent to deep foundations can impose downdrag loads. However, the majority of fine grained soils along the alignment are generally overconsolidated, and not prone to long-term settlement.

Downdrag loads can also be imposed by collapsible soils and settlements induced by seismic activity, consolidation, or potential localized subsidence. Refer to Sections 4.3.3, 4.4.4, 6.7.5, 6.7.6, and 6.7.8 for discussion on possible sources of settlements.

For bidding purposes, assume that any settlement associated with deep foundations in native soils for permanent structures will occur during construction and that long-term downdrag loads will be negligible. Should construction of adjacent fills or structures have the potential for inducing settlement and downdrag on deep foundations, these downdrag loads should be considered in design and controlled by construction means and methods.

Soils along the CP4 alignment are not considered sufficiently expansive to impose loads that require consideration in the design of deep foundations. For the purposes of bidding, assume uplift loads due to expansive soils do not need to be considered in the foundation design.

7.1.5 Pile Caps and Abutments

Potential scour at the HSR bridge/viaduct crossings is expected to be 15 to 35 feet (URS/HMM/Arup 2013d) for the main channels of the major rivers and creeks for a 100-year storm event, depending on the specific channel, flow, and bridge foundation dimension and configuration at each waterway. Scour countermeasures should be selected, designed, constructed, and maintained per the procedure and methods documented in *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition HEC 23* (FHWA 2009).

7.2 Retaining Walls

7.2.1 Wall Type Selection

Permanent retaining walls for approaches to HSR viaducts include conventional cast-in-place concrete walls and mechanically stabilized earth walls. Mechanically stabilized earth walls are also anticipated at bridge abutments for roadway overcrossings.

7.2.2 Structural Fill

Excavations required for HSR along CP4 are generally only for foundation preparation, with the exception of the permanent re-grading required to lower Kimberlina Road on the order of 20 feet.

Thus, there is little opportunity for recovering structural fill from foundation excavations or grading operations. Sources of fill could come from drainage/retention basins, but these volumes are not likely sufficient to balance the earthwork needs. For bidding purposes assume all structural fill for MSE Walls will be imported.

7.2.3 Lateral Deflections

Bespoke lateral retaining structures have not been considered because excavations associated with proposed improvements are expected to be too far from the existing structures that will remain in place.

The baseline ground conditions indicate that lateral deflections during excavations for temporary and permanent structures will be manageable. Utilities sensitive to lateral movements will require pre-excavation condition surveys. Threshold deflection values and response plans associated with excessive deflections will vary by structure and stakeholder requirements should there become a necessity for temporary excavations.

7.2.4 Drainage and Scour

The alignment crosses multiple floodplains including the Poso Creek, Shafter, and Weidenbach Street floodplains. Structures within floodplains should be adequately designed to facilitate drainage.

Adequate drainage is essential to the performance of retaining walls that are not designed for hydrostatic loads. Numerous structures along the alignment have provisions for hydraulic crossings. Structural backfill for retaining walls shall be free draining or protected from hydrostatic buildup using geocomposite drainage strips.

The only significant body of water that potentially could require special consideration for scour protection on the CP4 alignment is Poso Creek. Embedded foundations for these structures should consider the potential for scour if located inside the flow boundary. Design of deep foundations for scour protection shall be in accordance with the procedures provided by FHWA (2009) and other contract documents. Minimum embedment of permanent structure below ground surface shall be in accordance with applicable design standards for the given structure.

7.3 Embankments and At-Grade

7.3.1 Material Selection

Embankment materials consist of embankment fill, transition zone fills, structural fill, drainage layers, and geosynthetics.

Transition zone materials are required where embankments support trackway approach structures. Transition zone materials shall consist of structural fill mixed with cement, as required.

7.3.2 Subgrade Compressibility

The embankment foundation design must consider the potential for post-construction settlement both for static and dynamic conditions. Requirements for overexcavation or other remediation of soft or loose soils should be determined based on characterization of the subgrade and existing fill from future GIs to be carried out by the Contractor. Typical construction practice for embankment construction in areas of known existing fill is to excavate to firm or stable conditions and backfill with material meeting fill and compaction requirements. If firm and stable conditions cannot be reached economically, ground improvement may be necessary.

As noted in the FB GSHR (URS/HMM/Arup 2013c) there are numerous small irrigation ponds that can be seen on USGS quads but are not visible on current satellite imagery. The Contractor should make a complete inventory of these locations by examining all available maps and design a GI to determine their limits, and anticipate additional earthwork effort to muck out or treat the areas if necessary to achieve stable subgrades.

For bidding purposes, assume all existing fill is to be removed and replaced with suitable materials in accordance with the Contract Documents unless otherwise directed in other contract documents.

7.3.3 Compaction Control

The Contractor shall provide quality control measures to ensure compliance with specified requirements. Embankment foundation and subgrade preparation and the placement and compaction of fills shall be performed under the surveillance of a California-registered Geotechnical Engineer employed by the Contractor, as required by the Contract Documents.

7.3.4 Subgrade Preparation

Subgrade preparation includes fine grading, reworking as necessary, and preparation of cut, fill, or embankment upon which the structure and equipment foundations, pipe, sub-ballast, sub-base, base, and pavement will be placed. Unsuitable subgrade material, such as weak or compressible soils, shall be removed. The entire surface of subgrade shall be scarified, moisture conditioned, and recompacted in accordance with the Contract Documents. Subgrade stabilization material shall be incorporated if required.

7.3.5 Drainage, Scour, and Erosion

Where an embankment is located in a flood plain, the embankment design shall include slope protection consisting of a drainage layer and protection riprap. The drainage material shall be designed to comply with project design criteria. This layer should extend up to the highest flood water level plus additional freeboard as required by other contract documents and be underlain by a layer of geosynthetic membrane. Table 7.3-1 lists portions of the CP4 alignment intersected by mapped FEMA floodplains.

Table 7.3-1
Limits of FEMA 100-year Floodplains

Alignment	Floodplain Source	Limits of FEMA 100-yr Floodplain (Stations)
A1	Poso Creek	4713+00 to 4743+70, 4916+60 to beyond end of alignment
L1	Cross Creek	Before start of alignment to 5261+20
WS1	Shafter	5976+80 to 5995+80 5997+00 to 6031+40
WS1	Weidenbach St	6166+90 to 6263+20

Section 8.0

Construction Considerations

8.0 Construction Considerations

8.1 Regulatory Agencies

If temporary construction dewatering is utilized, a National Pollutant Discharge Elimination System permit from the Central Valley Regional Water Quality Control Board is required. In general, there is a long lead time required to obtain a National Pollutant Discharge Elimination System permit. Refer to the Contract Documents for Storm Water Pollution Prevention Plan requirements.

Gas detection and monitoring was not in the scope of the preliminary GI. It is the responsibility of the Contractor to investigate potentially gassy conditions that may be present during construction.

Trench excavations, shoring systems, sloped cuts, and other temporary structures shall comply with Occupational Safety and Health Administration (OSHA) 29 CFR 1926.650 and Caltrans regulations.

8.2 Site Constraints

The Contractor shall conduct a site review to identify site-specific constraints that will impact the selection of construction sequence, equipment, and methods. Items affecting the selection of construction means and methods include but are not limited to (1) site accessibility and space restrictions; (2) restrictions on traffic disruption; (3) environmental concerns, including local restrictions on construction noise, vibration, and dust; (4) easement and right-of-way restrictions; (5) railroad operations; (6) watercourses and irrigation infrastructure; (7) relocation(s) of critical area utilities; and (8) location(s) of overhead and underground utilities and nearby structures.

8.3 Corrosive Soils and Groundwater

Both laboratory soil corrosion and groundwater chemistry testing conducted for PE4P design and presented in the CP4 GDR (URS/HMM/Arup 2014) did not indicate the presence of a corrosive subsurface environment. However, assumptions regarding corrosive conditions for bidding purposes are provided in Section 6.5.

8.4 Contaminated Soils

The GI conducted for PE4P did not indicate the presence of contaminated soils. However, because the project alignment follows existing freeway and railroad corridors, portions of which are heavily industrialized, the Contractor shall expect to encounter surficially contaminated soils along these corridors during excavation and dispose of them in accordance with all regulatory requirements. No special consideration or baseline is set forth herein.

A soil management plan and site-specific health and safety plan must be implemented prior to initiation of construction activities. If evidence of contaminated soil is found during excavation activities (e.g., stained soil, odors), soil sampling and testing will be required prior to any disposal or reuse. Refer to the Contract Documents for more information.

8.5 Difficult Excavation

No hardpan layers were encountered. For baseline purposes assume conventional excavation and drilling equipment will be suitable.

8.6 Groundwater Inflows

The baseline unconfined groundwater table is below the depth of anticipated excavations made for subgrade preparation. However, due to extensive irrigation that occurs along the entire CP4 alignment, there is a potential for perched groundwater to be present during excavation and subgrade preparation operations. The presence of perched groundwater during excavation may reduce the stability of excavated slopes and create unwanted softening or heaving of soils at the base of the excavation.

Shallow perched groundwater conditions are possible in excavations made in the vicinity of the alignment, particularly near retention ponds. In the event that shallow or perched groundwater conditions exist, appropriate dewatering techniques should be employed. Likely dewatering systems consist of in-excavation sumps. Global dewatering schemes are not anticipated and shall be avoided due to potential impacts on adjacent structures, if applicable. To the extent practical, permanent retention facilities and other applicable drainage and stormwater facilities should be constructed in the early stages to serve as the discharge point for dewatering activities.

8.7 Track and Roadway Subgrade Improvement

Existing fill was encountered in a number of boreholes along the CP4 alignment during the PE4P GI. The Contractor shall anticipate variability in the thickness and suitability of existing fill for reuse. Deleterious material in the existing fill may include but is not limited to wood, glass, brick, metal, coarse gravel, and cobbles, and should be removed prior to reuse. Existing fill soils are likely suitable for reuse provided they satisfy quality requirements in terms of fines content, gradation, Atterberg limits, and electrochemical properties as required by the Contract Documents.

Soils along the alignment are relatively uniform and may be suitable for the proposed HSR track construction. However, unsuitable or saturated materials, such as soft clays, loose sands, and existing fills are likely present at shallow depths at some isolated locations in this area. The GI conducted for this preliminary design stage is inadequate to characterize the presence and extent of these areas. Some soil improvement measures, such as lime treatment or overexcavation and replacement with engineering fill materials, are likely to be needed to improve the subgrade during the track construction.

8.8 Utilities and Other Obstructions

The Fresno to Bakersfield 15% Record Set Utility Impact Report (URS/HMM/Arup 2013e) identifies 35 High Risk Utilities, numerous Low Risk Utilities, and 8 Special Utility Considerations. The Contractor is directed to this report for further information on the location and type of utilities at risk.

Existing utility information is provided in the Contract Documents, which include all known utilities such as the following:

- Overhead high-voltage transmission main relocations.
- Buried longitudinal utilities within freight rail rights-of-way where the freight rail trackage requires relocation to accommodate the HSR right-of-way.
- Gas mains.
- Fiber optic lines.

8.9 Deep Foundations

Deep foundations will be required to support the viaduct piers, retaining walls, and bridge abutments. This section discusses several issues that should be considered regarding anticipated deep foundation types for this project (CIDH piles and driven piles).

8.9.1 Driven Piles

Difficult driving conditions and predrilling are not anticipated along the CP4 alignment. However, piles may be subject to refusal if the hammer energy is too low to drive the pile. The Wave Equation Analysis of Piles can be used to help select the proper pile driving equipment and predict drivability of piles. Wave Equation Analysis of Piles simulates and analyzes the dynamics of a pile under hammer impacts according to one-dimensional elastic wave propagation theories. The results are used to predict the dynamic compatibility of the hammer, pile, and soil for evaluation of drivability of driven piles.

The Contractor shall select equipment to safely install the pile to the desired depth and capacity without damage.

8.9.2 Cast-in-Drilled-Hole Piles

CIDH piles can be advanced in this region using conventional techniques such as drilling an open dry hole, drilling the hole with water, drilling the hole with bentonite slurry, and drilling a temporarily cased hole. Each of these methods has its advantages and disadvantages. For baseline purposes, assume CIDH piles will require temporary support to prevent caving given the granular nature of the soils.

Cobbles and boulders can impede drilling operations. Cobbles and boulders were not encountered during the exploration. Discussion and baseline statements on potential debris/obstructions in fill and cemented soil conditions were provided in Section 6.2.

8.10 Excavations

Excavations will be required for the pile caps, footings, and subgrade preparation of at-grade and retained areas. Trenching may also be required for utility installation. Shallow excavations may be cut vertically if the soils will "stand up" without shoring, but only within the limits prescribed by OSHA and only under the supervision of a "competent person" as defined by OSHA and/or Cal/OSHA.

In some areas the soils may be too loose or granular to achieve a 5-foot excavation and a sloped cut or bracing must be used in conjunction with falsework and engineered backfill. Backfill at sloped pile cap excavations must be compacted to provide sufficient lateral resistance.

Surface runoff on the site should be controlled so that it does not flow into open excavations. Surface runoff shall conform to standard Storm Water Pollution Prevention Plan requirements.

8.11 Existing Features

Existing features along the CP4 alignment of interest include, but are not limited to, the following:

- BNSF Railroad.
- SR 43.
- Poso Creek.
- Pond-Poso Creek Fault.

- SR 46.
- Numerous irrigation canals.
- Oil wells.
- Retention ponds.

These and other features are crossed by the alignment and are to be considered in the design and construction of HSR facilities.

8.12 Environmental Concerns

Noise and vibrations produced through the construction of the project structures should adhere to the project's environmental management plan and comply with state and federal health and safety regulations.

Construction schedules shall consider earthwork to take advantage of the dry season (April through October). Earthwork in the dry season must include provisions for dust mitigation in accordance with local and regional air quality regulations. Dust in the SJV is known to contain spores that cause Valley Fever. Dust control will be of paramount importance.

Requirements for erosion control are found in Specification Section 31 05 00. Other environmental concerns may be found in the FB EIR/EIS (URS/HMM/Arup 2013b).

Refer to Contract Documents for project background and requirements regarding environmental impacts and mitigation strategies.

8.13 Archeological and Historic Environmental Resources

As a result of the studies conducted in support of the FB EIR/EIS (URS/HMM/Arup 2013b), seven archaeological sites were identified within the project alignments. None of these sites were considered significant and thus do not warrant additional treatment or mitigation (see *California High-Speed Train Fresno to Bakersfield Archaeological Survey Report*). However, due to limitations in permission to enter, only approximately 20% of the HSR project alignment footprint has been subject to archaeological survey. In addition, a number of areas were identified that will require additional investigations and potentially require monitoring during construction. These future studies will be conducted per the stipulations of the Section 106 Programmatic Agreement and the Archaeological Treatment Plan and Memorandum of Agreement. These documents will define the process by which these treatment measures will be applied to each known resource and will outline measures for the phased identification of historic properties as additional parcel access is obtained and design work is completed.

A number of significant historic architectural resources have been identified within the HSR project footprint (see California High-Speed Train Fresno to Bakersfield Historic Architectural Survey Report, California High-Speed Train Fresno to Bakersfield Historic Property Survey Report, and supplements prepared in 2013 and 2014). As with archaeological resources, in addition to the mitigation measures provided in the EIR/EIS, a series of treatment measures will be formulated per the stipulations of the Section 106 Programmatic Agreement and the Built Environment Treatment Plan and Memorandum of Agreement. These documents will define the process by which these treatment measures will be applied to each known resource and will outline measures for the phased identification of historic properties as additional parcel access is obtained and design work is completed.

8.14 Geotechnical Permitting

Geotechnical explorations must be conducted during the design-build phase of the project to augment the geotechnical data collected during PE4P. Geotechnical exploration permitting generally falls in two categories: (1) permits to drill within riparian areas and (2) permits to drill outside riparian areas. Drilling permits for areas outside of riparian habitats are typically obtained from city and county environmental health agencies.

Permits to encroach on jurisdictional rights-of-way should be obtained from the local agency, county, or Caltrans, as appropriate.

8.15 Construction Consideration Matrix

Table 8.15-1 has been prepared to capture the site conditions that would be of concern to a bidding contractor, from an engineer's perspective. The list is not exhaustive but identifies some conditions at each of the planned structures that could have cost implications when considered as part of the bid preparation.

Table 8.15-1
Construction Considerations Matrix

Location	Approximate Start Station (ft)	Approximate End Station (ft)	Track Subgrade Improvement	Hydraulic Crossing	Shallow Groundwater	Difficult Excavation - Hardpan	Buried Utilities and Other Obstructions	Compressible Soil	Expansive Soil	Collapsible Soil	Potential for Flooding	Corrosive Soil	Regulatory Agencies	Deep Foundations	Protection of Existing Structures / Utilities	Unforeseen Ground Conditions	Need for Additional Geotechnical Data
At-Grade 1	4435+50	4925+51	X	X	X		X	X					X			X	X
At-Grade 2	5154+50	5191+50	X				X	X					X			X	X
Retained Embankment 1	5191+50	5225+40		X				X			X		X			X	X
Structure 1	5225+40	5227+80		X			X				X		X	X		X	X
Retained Embankment 2	5227+80	5271+60											X			X	X
At-Grade 3	5271+60	5322+33	X	X			X						X			X	X
At-Grade 4	5422+50	5551+00	X	X			X						X		X	X	X
Retained Embankment 3	5551+00	5556+40											X		X	X	X
Structure 2	5556+40	5557+60					X						X	X	X	X	X
Retained Embankment 4	5557+60	5564+80											X		X	X	X
Structure 3	5564+80	5682+95					X						X	X		X	X
Retained Embankment 5	5682+95	5709+50											X			X	X
At-Grade 5	5709+50	5716+02	X				X						X			X	X
Structure 4	5716+02	5716+70					X						X	X	X	X	X
At-Grade 6	5716+70	5928+55	X				X						X			X	X
Retained Embankment 6	5928+55	5955+30											X		X	X	X
Structure 5	5955+30	6117+25					X						X	X	X	X	X
Retained Embankment 7	6117+25	6151+00											X		X	X	X
At-Grade 7	6151+00	6292+50	X				X						X		X	X	X

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Section 9.0

Instrumentation and Monitoring

9.0 Instrumentation and Monitoring

The design criteria mandate specific limits on the total and differential settlements of embankments, transition zones, and abutments. This will require accurate measurements be made. Moreover, subsidence rates along the alignment are ongoing. Thus, establishing an early array of surface settlement monuments and a periodic monitoring program early in the contract to verify the subsidence rates could be a critical element of the Contractor's design. Refer to the Contract Documents for specific instrumentation and monitoring requirements.

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Section 10.0

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10.0 References

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Section 11.0

Glossary

11.0 Glossary

Atterberg limits: The water contents of a soil mass corresponding to the transition between a solid, semi-solid, plastic solid, or liquid. Laboratory test used to distinguish the plasticity of clay and silt particles.

Boulder: A rock fragment that will not pass through a 12-inch (305-millimeter) square opening, no matter how it is oriented in the opening. Boulder sizes are defined by the smallest size opening that the boulder can be oriented to pass through.

Bulking/swell factor: $\frac{\text{volume of soil after excavation}}{\text{volume of soil in situ}}$.

Cobbles: Soil particles between 3 inches (76 millimeters) and 12 inches (305 millimeters) in size.

Cohesion: The force that holds together molecules or like particles within a substance.

Cohesionless soils: Granular soils (silt, sand, and gravel type) with no shear strength unless confined.

Cohesive soils: Contains clay minerals and possesses plasticity.

Consolidation: Reduction in soil volume due to squeezing out of water from the pores as the soil comes to equilibrium with the applied loads.

Dewatering: The removal of groundwater to reduce the flow rate or diminish water pressure. Dewatering is usually done to improve conditions in surface excavations and to facilitate construction work.

Dry unit weight: The weight of solids (soil grains) to the total unit volume of soil. Units lb/ft^3, kN/m^3.

Firm, firm ground: Soil that remains stable in walls and face of an opening without initial support for sufficient time to permit installation of final support.

Flowing, flow, flowing ground: Soil that moves like a viscous liquid into an excavation.

Grain size distribution, particle size distribution: Soil particle sizes that are determined from a representative sample of soil that is passed through a set of sieves of consecutively smaller openings.

Groundwater: Water that infiltrates into the earth and is stored in the soil and bedrock within the zone of saturation below the earth's surface.

Hydrostatic head, hydrostatic pressure, pressure head: The height of a column of water required to develop a given pressure at a given point. Head may be measured in either height (feet or meters) or pressure (pounds per square inch, kilograms per square centimeter, or bars).

Natural water content: The ratio between the mass of water and the mass of soil solids. w = (total unit weight – dry unit weight) / dry unit weight.

Normalized cone resistance (Q_t): CPT tip resistance in a non-dimensional form and taking account of the in situ vertical stresses.

$$Q_t = (q_t - \sigma_{v0})/\sigma_{v0}'$$

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Normalized friction ratio (F_r): The ratio, expressed as a percentage, of the sleeve friction (f_s) to the cone resistance (q_t) taking account of the in situ vertical stresses.

$$F_r\,(\%) = \left(\frac{f_s}{q_t - \sigma_{v0}}\right)100 F_r\,(\%) = [f_s / (q_t - \sigma_{vo})]100$$

Normalized CPT soil behavior type (SBT_N): Soil behavior type based on normalized cone resistance (Q_t) and normalized friction ratio (F_r).

Normally consolidated: A soil where the current effective overburden pressure is equal to the maximum overburden pressure.

Perched groundwater: An unconfined groundwater body in a generally limited area above the regional water table and separated from it by a low-permeability, unsaturated zone of bedrock or soil.

Permeability: The capacity of bedrock or soil to permit fluids to flow through it.

q_c: CPT cone resistance.

q_t: CPT cone resistance corrected for pore water effects, where A_n is the cone tip area ratio:

$$q_t = q_c + u_2(1 - A_n)$$

Raveling, slow raveling, fast raveling: Chunks or flakes of material drop out of the excavated surface due to loosening or to overstress and "brittle" fracture. In fast raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.

Regional subsidence: Large-scale, slow-occurring, typically unnoticeable deformation of the ground surface attributable to tectonic activity, groundwater abstraction, or extraction of other liquids or gasses. Typical magnitudes of regional subsidence are on the order of inches or feet occurring over decades across tens of miles.

Running, cohesive running ground: Granular soils that move freely into the excavated area. Granular materials without cohesion are unstable at a slope greater than their angle of repose. When exposed at steeper slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose. Cohesive running ground exhibits some apparent cohesion that exists from moisture content, weak cementation, and overconsolidation.

Shear strength: The maximum shear stress that a soil can sustain under a given set of conditions. For free-draining cohesionless soil, shear strength is generally modeled by the Mohr-Coloumb failure criteria, which approximates shear strength as the product of the effective stress and the tangent of the angle of internal friction. For cohesive soil, shear strength under drained conditions can be represented as per cohesionless soil. For undrained conditions, the shear strength of cohesive soil is generally represented by an undrained shear strength parameter.

Shrinkage factor: $\frac{\text{volume of soil after compaction}}{\text{volume of excavated soil before compaction}}$

Specific gravity: The ratio of the density of a body or a substance to the mass of an equal volume of water.

Standard penetration test, N-value: Field test performed in general accordance with ASTM D 1586, Test Method for Penetration Test and Split – Barrel Sampling of soils. Test involves driving a 2-inch OD, 1.375 inch ID, split spoon sampler with a 140-pound hammer, falling freely from a height of 30 inches. The number of blows required to achieve each of three 6-inch increments of

sampler penetration is recorded. The density of cohesionless or coarse-grained soils, and relative consistency of cohesive or fine-grained soils is defined as below:

Cohesionless Soils		Cohesive Soils	
N, SPT Blows/ft	**Relative Density**	**N, SPT Blows/ft**	**Relative Consistency**
0–4	Very loose	Under 2	Very soft
4–10	Loose	2–4	Soft
10–30	Medium dense	4–8	Medium stiff
30–50	Dense	8–15	Stiff
Over 50	Very dense	15–30	Very stiff
		Over 30	Hard

Structural fill: Soils used as fill, such as retaining wall backfill, foundation support, dams, and slopes that are to be placed in accordance to engineered specifications. These specifications may delineate soil grain-size, plasticity, moisture, compaction, angularity, and other index properties depending on the application.

Swelling, swelling ground: Soil that undergoes a volumetric expansion resulting from the addition of water. Swelling ground may appear to be stable when exposed, with the swelling developing later. Ground absorbs water, increases in volume, and expands slowly. Increase in soil volume; volumetric expansion of particular soils due to changes in water content.

Total Unit Weight: Ratio between the total weight of soil including water and the total volume of the soil.

u_2: Pore pressure generated during cone penetration and measured by a pore pressure sensor just behind the cone.

Unconsolidated: Loose sediment, lacking cohesion or cementation.

Unified Soil Classification System (USCS): A system of soil classification based on grain size, liquid limit, and plasticity of soils.

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Appendix A
Soil Parameter Interpretations

Table of Contents

A1.0 Introduction

This appendix presents the results of data analyses undertaken to assist development of the baseline soil parameters presented in Section 6 of the main report.

The purpose of this appendix is to present the variability of baselined soil properties and parameters associated with the ground conditions encountered during the ground investigation. Histograms and cumulative distributions have been prepared to present the range, mean, median, and standard deviation of data collected during this ground investigation. These interpretations are provided to illustrate the uncertainty associated with the estimates of baseline soil parameters.

The appropriateness of the data presented herein have been reviewed, and in some cases, outlier data was excluded from interpretations. Correlations used to estimate soil parameters have been restricted to maximum values considered reasonable based on engineering judgment.

Soil parameters have been measured and interpreted following TM 2.9.10 *Geotechnical Analysis and Design Guidelines*, in general accordance with Geotechnical Engineering Circular No. 5 (FHWA 2002) and AASHTO LRFD Bridge Design (2010) recommendations.

Cone penetration test (CPT) interpretations were based primarily on correlations presented in Lunne (1997). In addition, CPT data collected during the investigations was analyzed using the commercially available software CPeT-IT v1.7.6.42, developed by Geologismiki.

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A2.0 CPT and Drilling Correlations

A2.1 Total Unit Weight

A2.1.1 CPT Correlation

Total unit weight was estimated from CPT results using the following correlation presented in Lunne (1997):

Table A2.1-1
Unit Weight by SBT, from CPT Data

SBT [a]	SBT Description	Unit Weight, γ_t (psf)
1	Sensitive fine grained	111.4
2	Organic soil	79.6
3	Clay	111.4
4	Silty clay to clay	114.6
5	Clayey sitl to silty clay	114.6
6	Sandy silt and clayey silt	114.6
7	Silty sand and sandy silt	117.8
8	Sand and silty sand	120.9
9	Sand	124.1
10	Sand to gravelly sand	127.3
11	Very stiff fine grained [b]	130.5
12	Sand to clayey sand [b]	120.9

[a] SBT uses an earlier interpretive method for soil behavior type by Robertson et al (1986). Note that the main report often referes to SBT_N, a normalized method developed by Robertson (1990) and revised (2010).
[b] Heavily overconsolidated and/or cemented

A2.2 Undrained Shear Strength

A2.2.1 CPT Correlation

Undrained shear strength was estimated from CPT results using the following correlation presented in Lunne (1997):

$$s_u = \frac{q_c - \sigma_{vo}}{N_k}$$

Where:

q_c = Measured cone resistance

σ_{vo} = (Total) Vertical overburden stress

N_k = Cone factor; taken as 17 for non-fissured overconsolidated clay

A2.3 Effective Friction Angle

A2.3.1 CPT Correlation

Effective fiction angle was estimated from CPT results using the following correlation presented in FHWA GEC No. 5 (after Robertson, 1983):

$$\Phi' = \arctan\left[0.1 + 0.38\, log\left(\frac{q_t}{\sigma'_{vo}}\right)\right]$$

Where:

σ'_{vo} = Effective vertical overburden stress

$q_t = q_c + u_2(1 - a)$ = Corrected cone resistance

u_2 = Pore pressurement measreument behind cone

a = Net cone area ratio (0.80 for site equipment used)

A2.3.2 Drilling Correlation

Effective fiction angle was estimated from SPT results using the following correlation presented in FHWA GEC No.5 (after Hatanaka and Uchida, 1996):

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20°$$

Where:

$(N_1)_{60}$ = SPT N-value corrected for overburden and field procedures (Section A2.4.2)

A2.4 Standard Penetration Test Blow Count

A2.4.1 CPT Correlation

SPT N_{60} was estimated from CPT results using the following correlation used in CPeT-IT v1.7.6.42:

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268 - 0.2917 I_c}}$$

Where:

I_c = Soil Behavior Type Index

Given By:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

Where:

Q_t = Normalized cone penetration resistance

F_r = Normalized Friction Ratio

A2.4.2 Drilling Correction

The SPT correction for field procedures (energy) was applied as follows:

$$N_{60} = C_E N_{SPT}$$

Where:

N_{SPT} = Uncorrected field SPT N-value. Where a modified California sampler was used, the following correlation was used: $N_{SPT} = 0.64 N_{MC}$

C_E = Correction factor for Energy Ratio (ER) as measured in the field = ER/60

The SPT correction for overburden was applied as follows:

$$(N_1)_{60} = C_N N_{60}$$

Where:

N_{60} = SPT N-value corrected for hammer energy

C_N = Stress normalization parameter calculated as $C_N = \left[0.77 \log\left(\frac{40}{\sigma'_v}\right)\right] \leq 2.0$

A2.5 Cone Tip Resistance

The measured cone resistance used for the statistical analyses refers to the static cone resistance q_c recorded during cone penetration testing, as follows:

$$q_c = \frac{Q_c}{A_c}$$

Where:

Q_c = Force acting on the cone

A_c = Projected area of the cone

A2.6 Soil Modulus

A2.6.1 CPT Correlation

For coarse-grained material, soil modulus was estimated from CPT results using the following correlation (after AASHTO 2010):

$$E_s = 4q_c$$

For fine-grained material, soil modulus was estimated from undrained shear strength using the equation below.

$$E_u = 300s_u$$

Where:

E_u = Undrained soil modulus of fine grained soil

s_u = Undrained shear strength, estimated from CPT data as per Section A2.2

A2.6.2 Drilling Correlation

For coarse-grained material, soil modulus was estimated from SPT results using the elastic constant for Category 2, indicated in Table A2.6-1 (after AASHTO 2010).

Table A2.6-1
SPT Correlation to Soil Modulus by Soil Type

Category	Soil Type	Soil Modulus (tsf)
1	Silt, sandy silts, slightly cohesive mixtures	$4(N_1)_{60}$
2	Clean fine to medium sands and slightly silty sands	$7(N_1)_{60}$
3	Coarse sands and sands with little gravel	$10(N_1)_{60}$
4	Sandy gravels and gravels	$12(N_1)_{60}$

Fine grained soils generally comprise very stiff overconsolidated mixtures of clay and silt are not applicable to Category 1. Estimation of soil modulus for fines using SPT N was not undertaken. Refer to CPT correlation above and further discussion in the main report.

A3.0 CP4 Soil

The following sections present the results of statistical analysis performed on data obtained from boreholes and CPTs within CP4.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 20 feet of soils and (2) soils below 20 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, drilling, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A3.1 Total Unit Weight

Table A3.1-1
Statistical Summary of Total Unit Weight – CP4

Total Unit Weight	CPT				Drilling*			
	Fine		Coarse		Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	258	4087	680	2676	9	42	16	43
Mean, pcf	116	120	120	122	121	127	123	127
Median, pcf	115	115	121	124	124	129	123	130
Standard Deviation, pcf	6	7	3	3	8	8	9	11
Maximum, pcf	131	131	131	131	130	137	137	137
Minimum, pcf	80	111	111	111	107	95	106	95
* Unit weight from drilling determined from samplers with full recovery.								

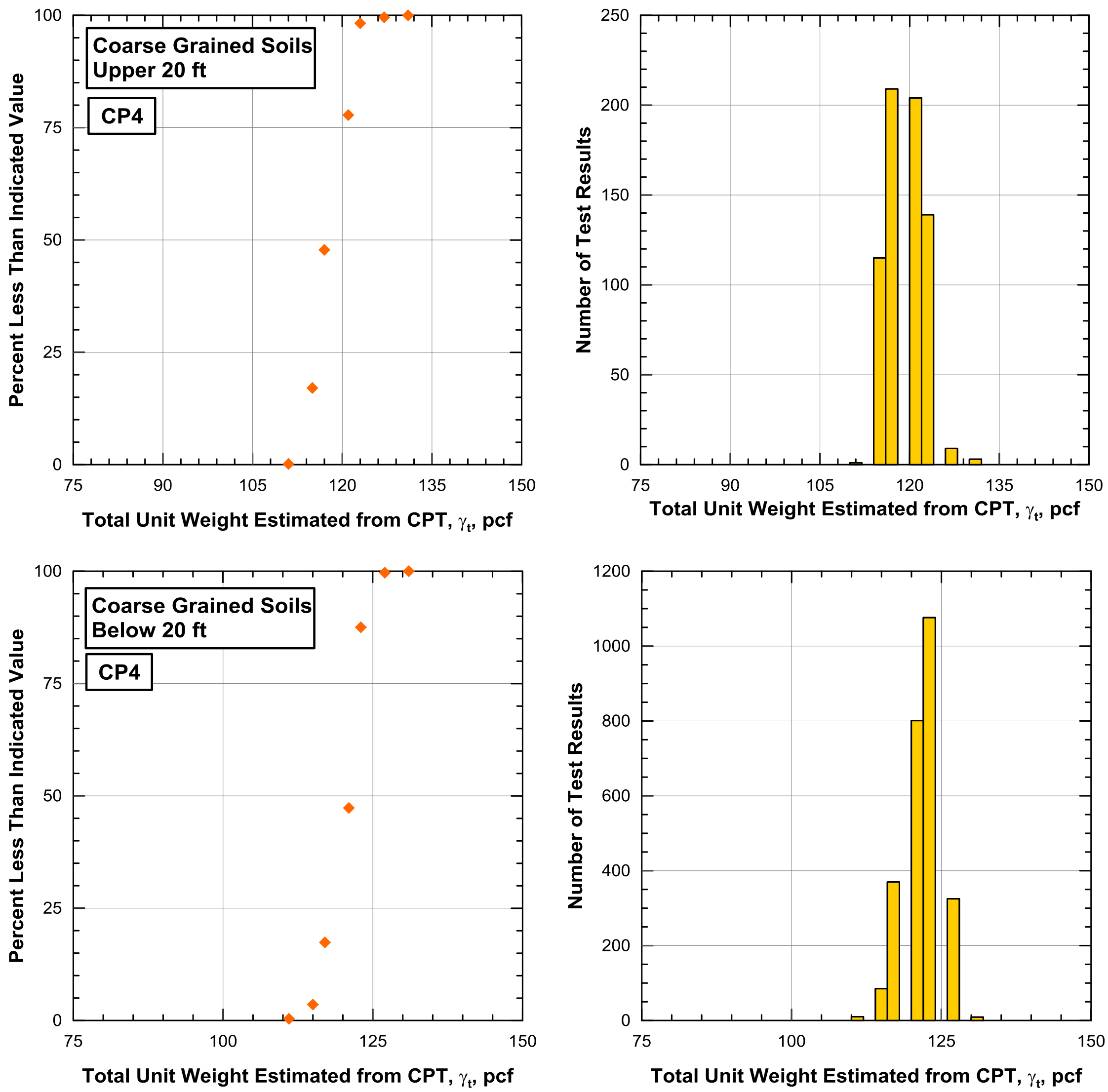

Figure A3.1-1
Statistical Summary of Total Unit Weight Estimated from CPT Data for Coarse Grained Soils – CP4

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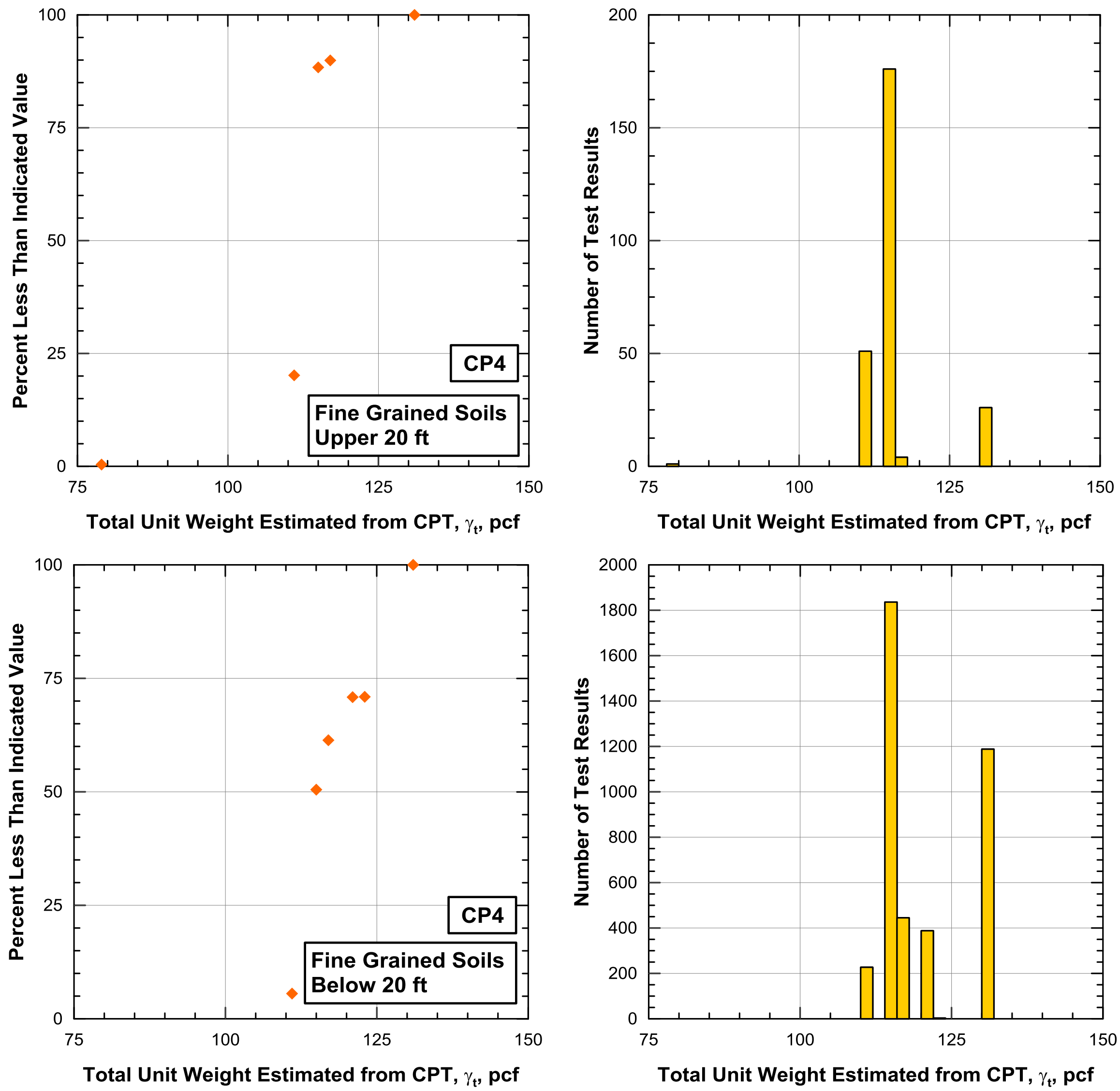

Figure A3.1-2

Statistical Summary of Total Unit Weight Estimated from CPT Data for Fine Grained Soils – CP4

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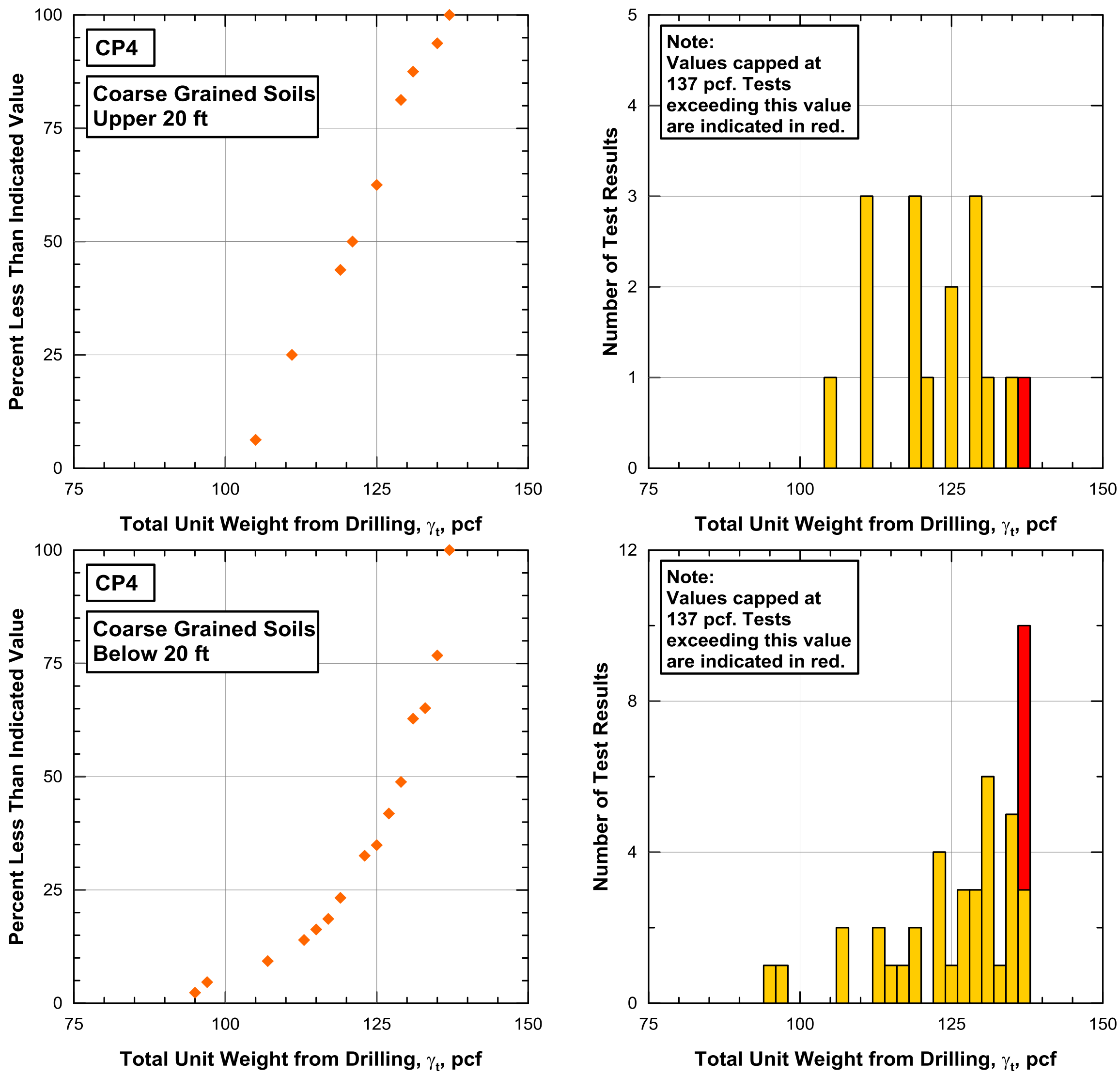

Figure A3.1-3
Statistical Summary of Total Unit Weight from Laboratory Results for Coarse Grained Soils – CP4

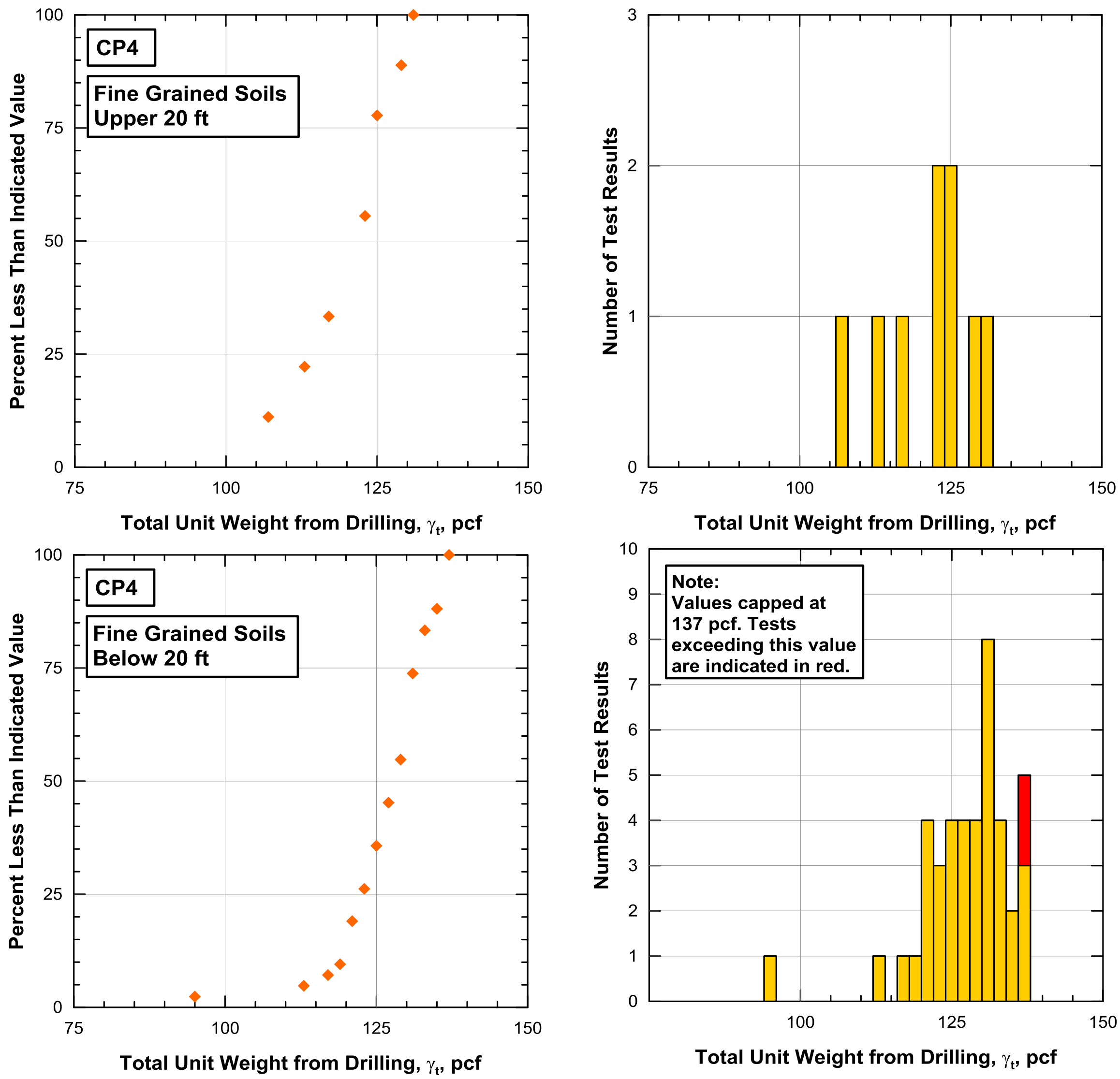

Figure A3.1-4
Statistical Summary of Total Unit Weight from Laboratory Results for Fine Grained Soils – CP4

A3.2 Effective Cohesion

Table A3.2-1
Statistical Summary of Effective Cohesion – CP4

Effective Cohesion	Laboratory			
	Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	-	8	16	29
Mean, psf	-	674	554	665
Median, psf	-	645	660	750
Standard Deviation, psf	-	339	381	345
Maximum, psf	-	1000	1000	1000
Minimum, psf	-	0	0	0

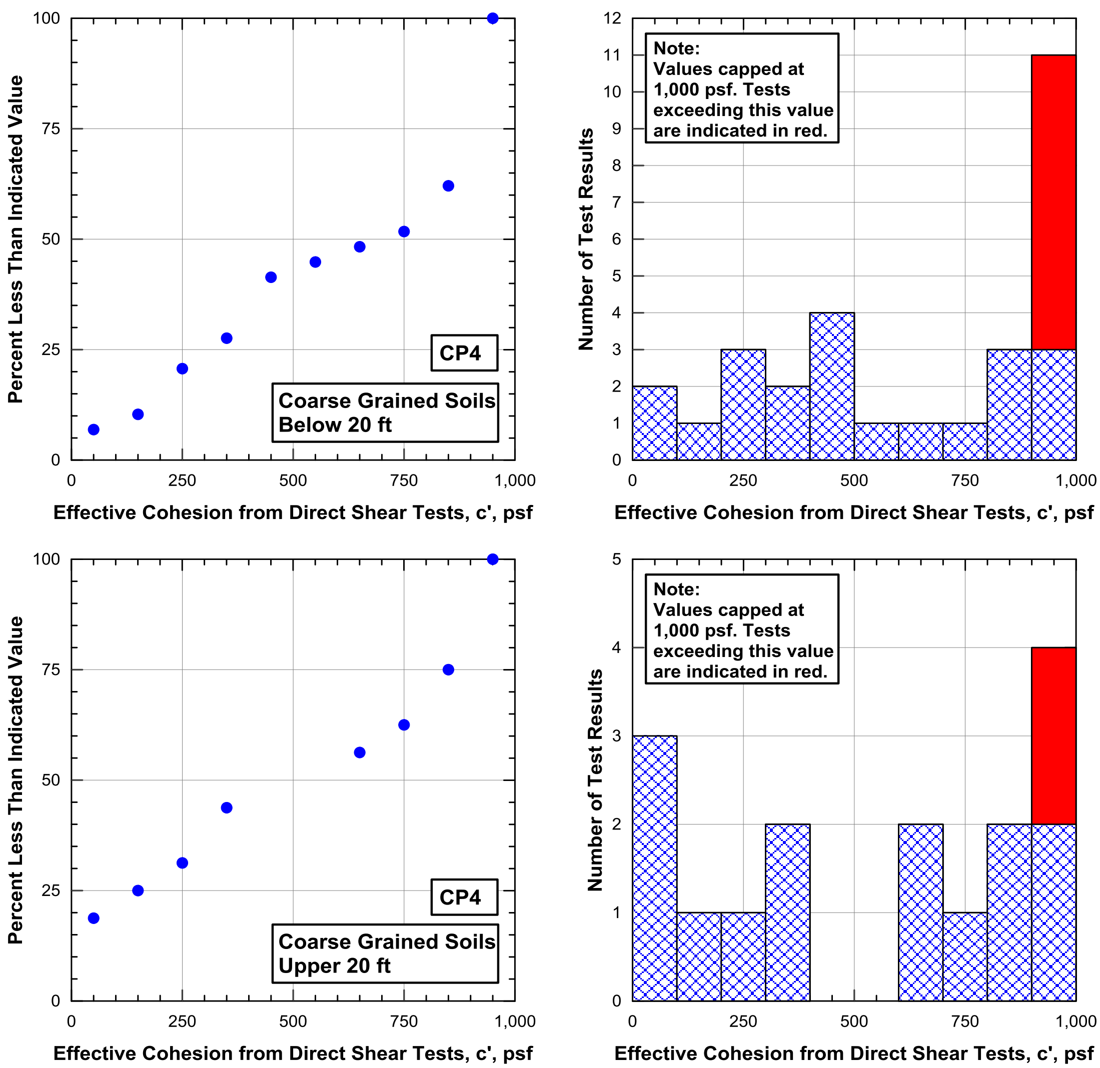

Figure A3.2-1
Statistical Summary of Effective Cohesion from Laboratory Results for Coarse Grained Soils – CP4

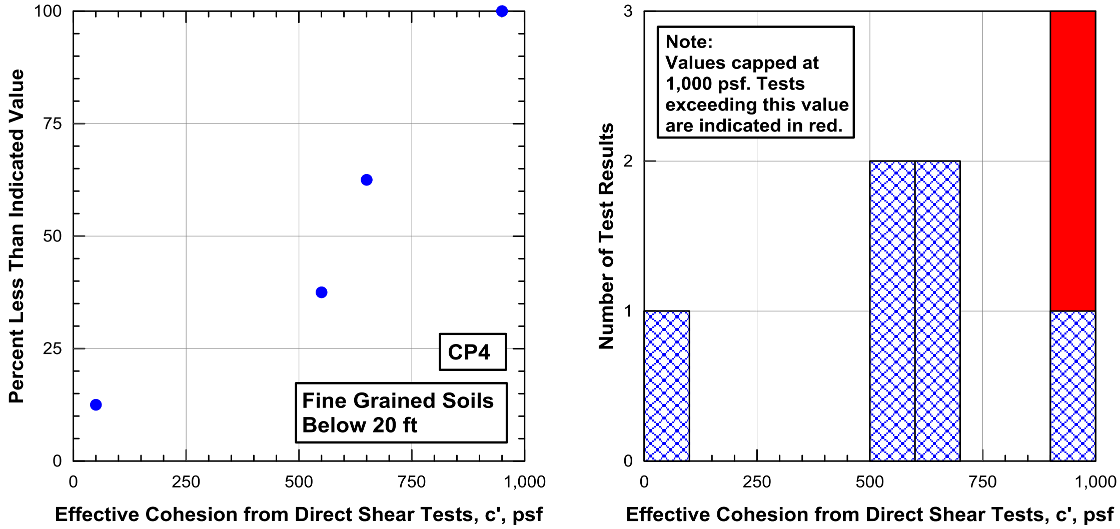

Figure A3.2-2

Statistical Summary of Effective Cohesion from Laboratory Results for Fine Grained Soils – CP4

A3.3 Effective Friction Angle

Table A3.3-1
Statistical Summary of Effective Friction Angle for CPT Data – CP4

Effective Friction Angle	CPT			
	Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	–	–	2780	10636
Mean, deg	–	–	41	40
Median, deg	–	–	41	40
Standard Deviation, deg	–	–	3	2
Maximum, deg	–	–	50	46
Minimum, deg	–	–	30	29

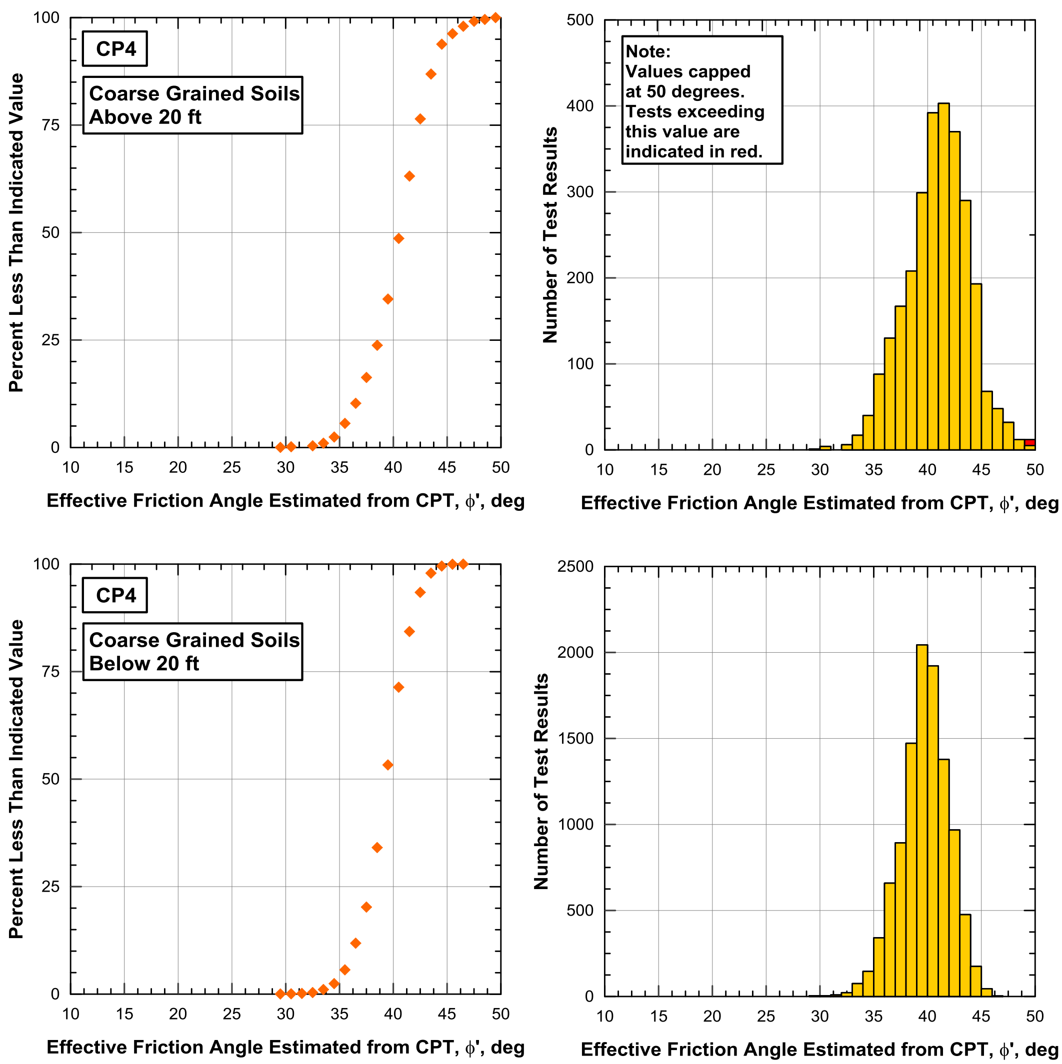

Figure A3.3-1
Statistical Summary of Effective Friction Angle Estimated from CPT Data for Coarse Grained Soils– CP4

Table A3.3-2
Statistical Summary of Effective Friction Angle for Drilling Data – CP4

Effective Friction Angle	Drilling		Laboratory			
	Coarse		Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	41	248	-	8	16	29
Mean, deg	38	44	-	29	31	33
Median, deg	38	44	-	28	31	33
Standard Deviation, deg	5	4	-	5	5	4
Maximum, deg	50	50	-	38	37	42
Minimum, deg	30	36	-	23	21	23

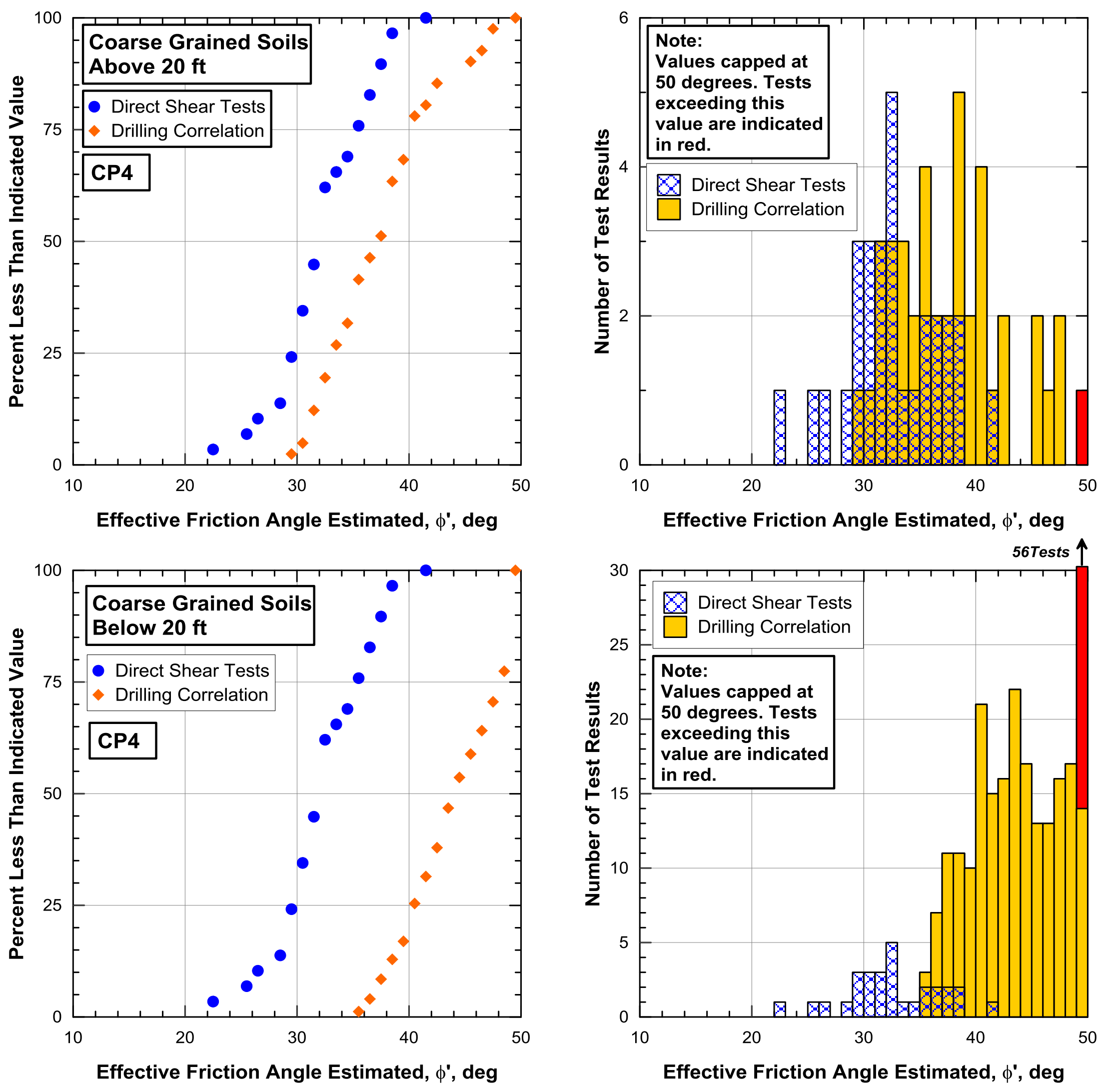

Figure A3.3-2
Statistical Summary of Effective Friction Angle Estimated from Drilling and Laboratory Data for Coarse Grained Soil – CP4

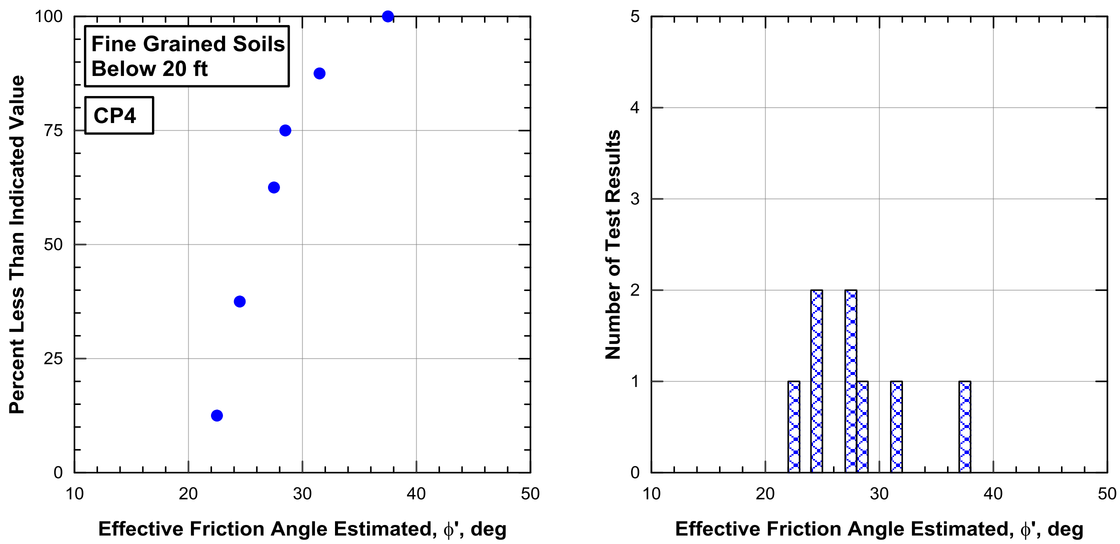

Figure A3.3-3
Statistical Summary of Effective Friction Angle Estimated from Laboratory Data for Fine Grained Soil – CP4

Table A3.3-3
Statistical Summary of Effective Friction Angle for Drilling Data – CP4

		Borehole ID	Depth (ft)	USCS	Results By Test φ'	Least-Squares Trendline by Soil Type φ'
Above	**Fine**	S0075R	6	ML	36	31
		S0079R	16	ML/SM	34	
Below	**Fine**	S0077R	46	CL	28	29
		S0077R	82	CL	26	
		S0077R	106	ML	28	
		S0078R	73	ML	18	
		S0079R	68	CL	31	
		S0084R	41	ML	35	
		S0084R	41	ML	35	
		S0088R	92	CL	34	
		S0088R	92	CL	32	
	Coarse	S0085R	86	SC	33	36
		S0085R	86	SC	35	
		S0088AR	41	SM	40	
		S0088AR	41	SM	38	
		S0088R	51	SM	37	
		S0088R	51	SM	37	
		S0091R	100.8	SC	36	

A3.4 SPT N_{60}

Table A3.4-1
Statistical Summary of SPT N60 – CP4

SPT N_{60}	CPT				Drilling			
	Fine		Coarse		Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	1039	16382	2780	10636	20	154	40	248
Mean, blows/ft	14	41	22	71	22	47	18	69
Median, blows/ft	13	36	20	74	22	42	17	72
Standard Deviation, blows/ft	8	21	11	25	8	24	11	26
Maximum, blows/ft	71	99	72	99	37	99	47	99
Minimum, blows/ft	3	4	3	10	5	12	4	17

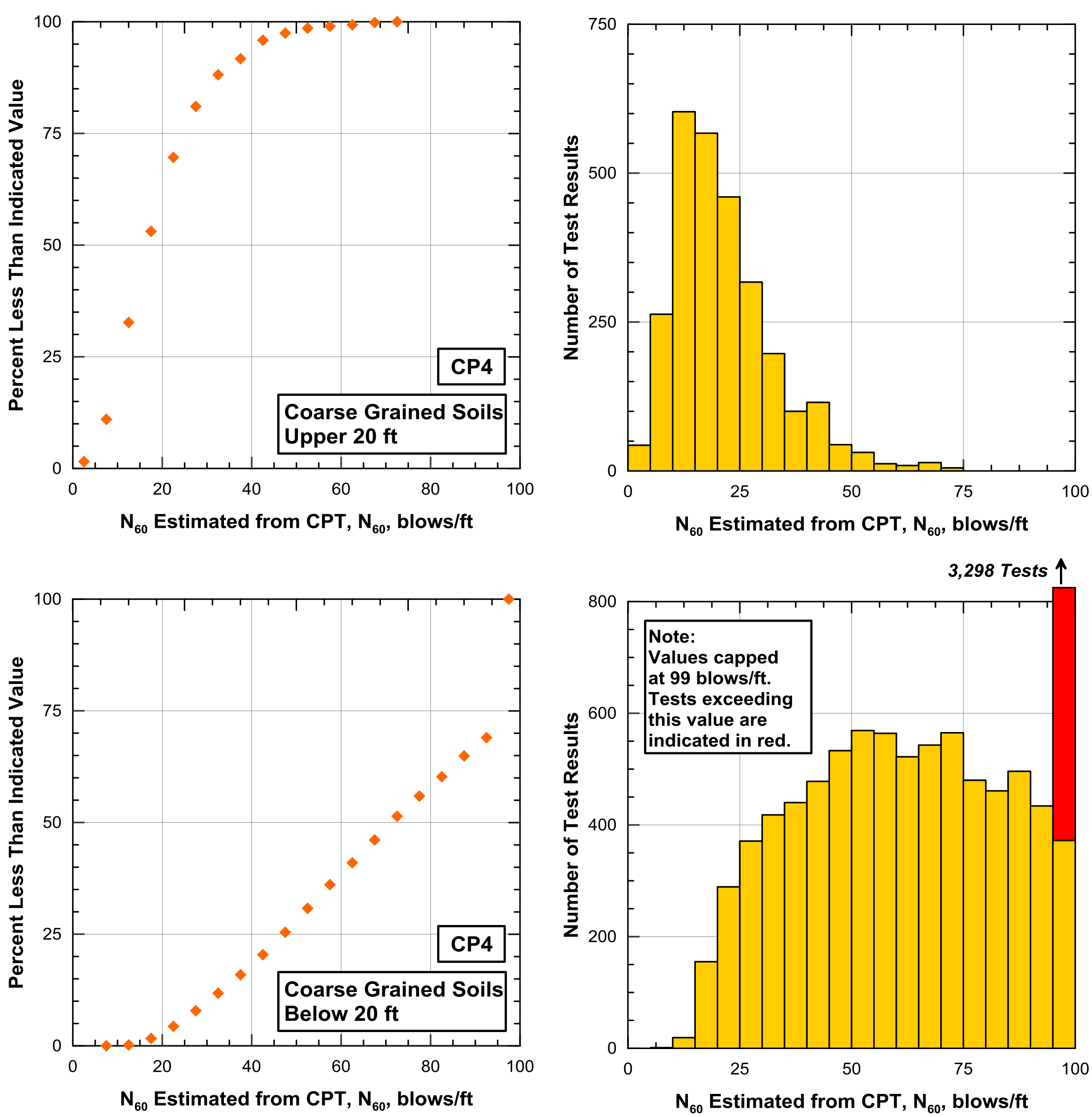

Figure A3.4-1
Statistical Summary of SPT N_{60} Estimated from CPT Data for Coarse Grained Soils – CP4

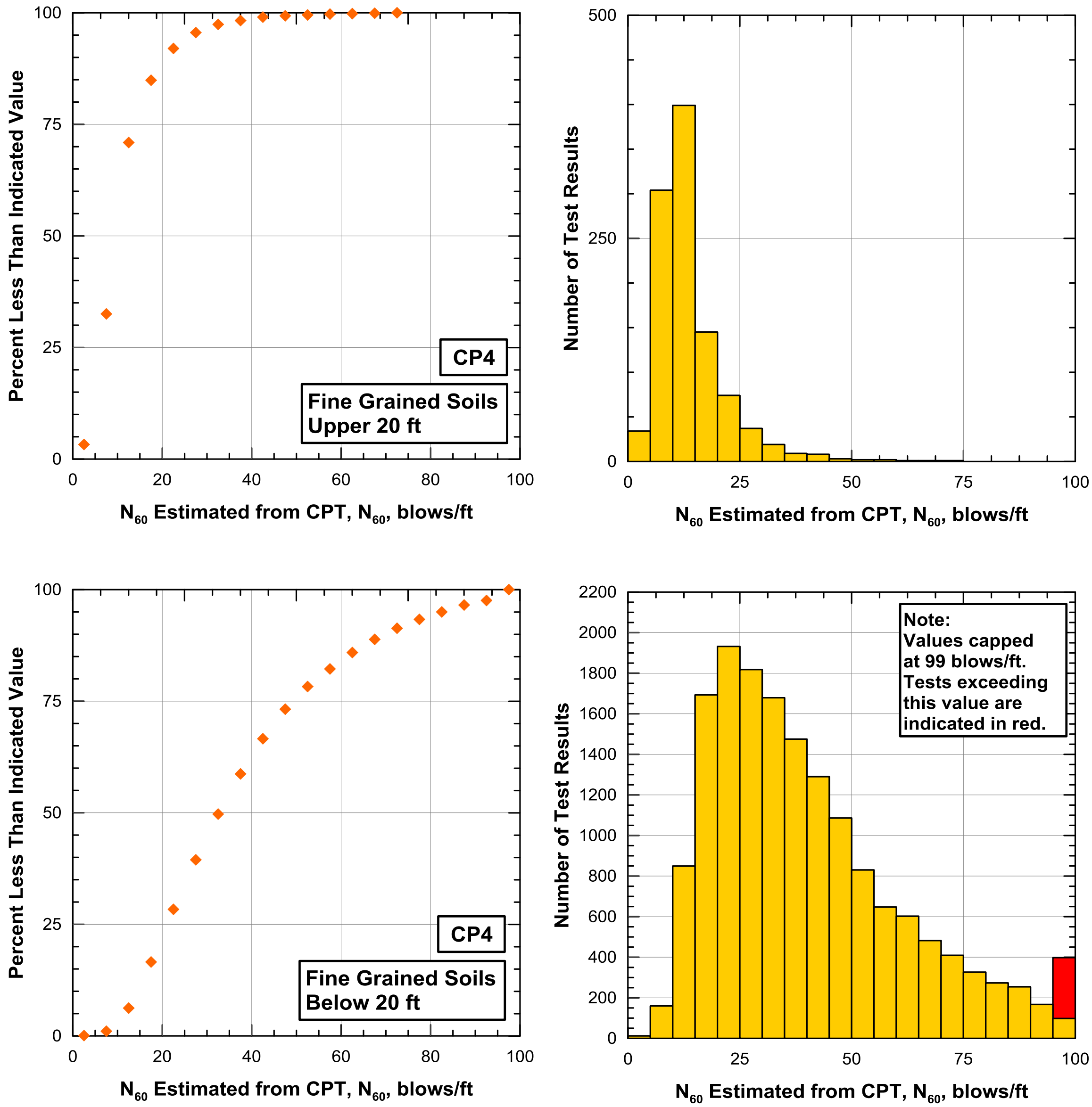

Figure A3.4-2
Statistical Summary of SPT N_{60} Estimated from CPT Data for Fine Grained Soils – CP4

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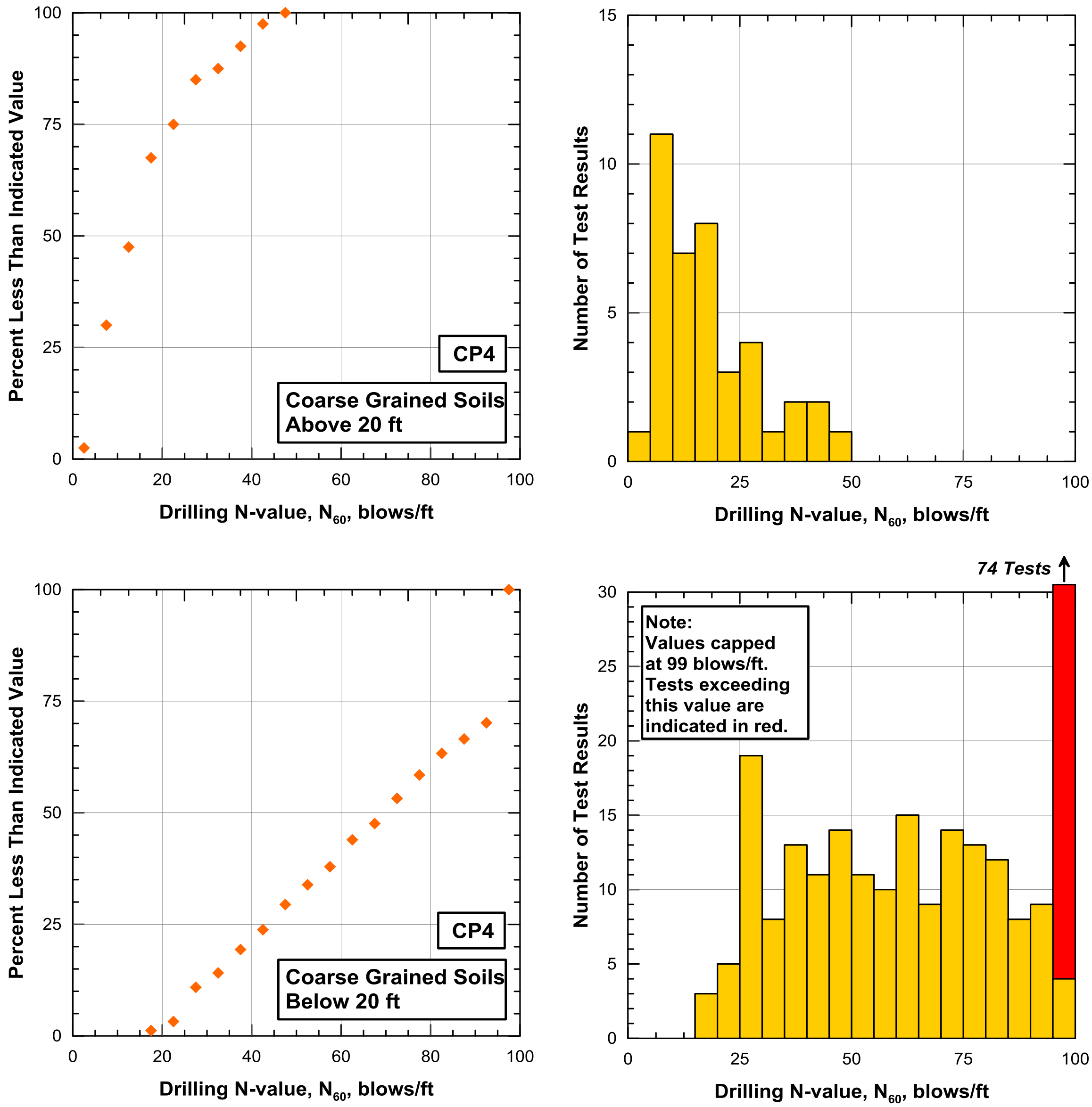

Figure A3.4-3
Statistical Summary of SPT N_{60} for Coarse Grained Soils – CP4

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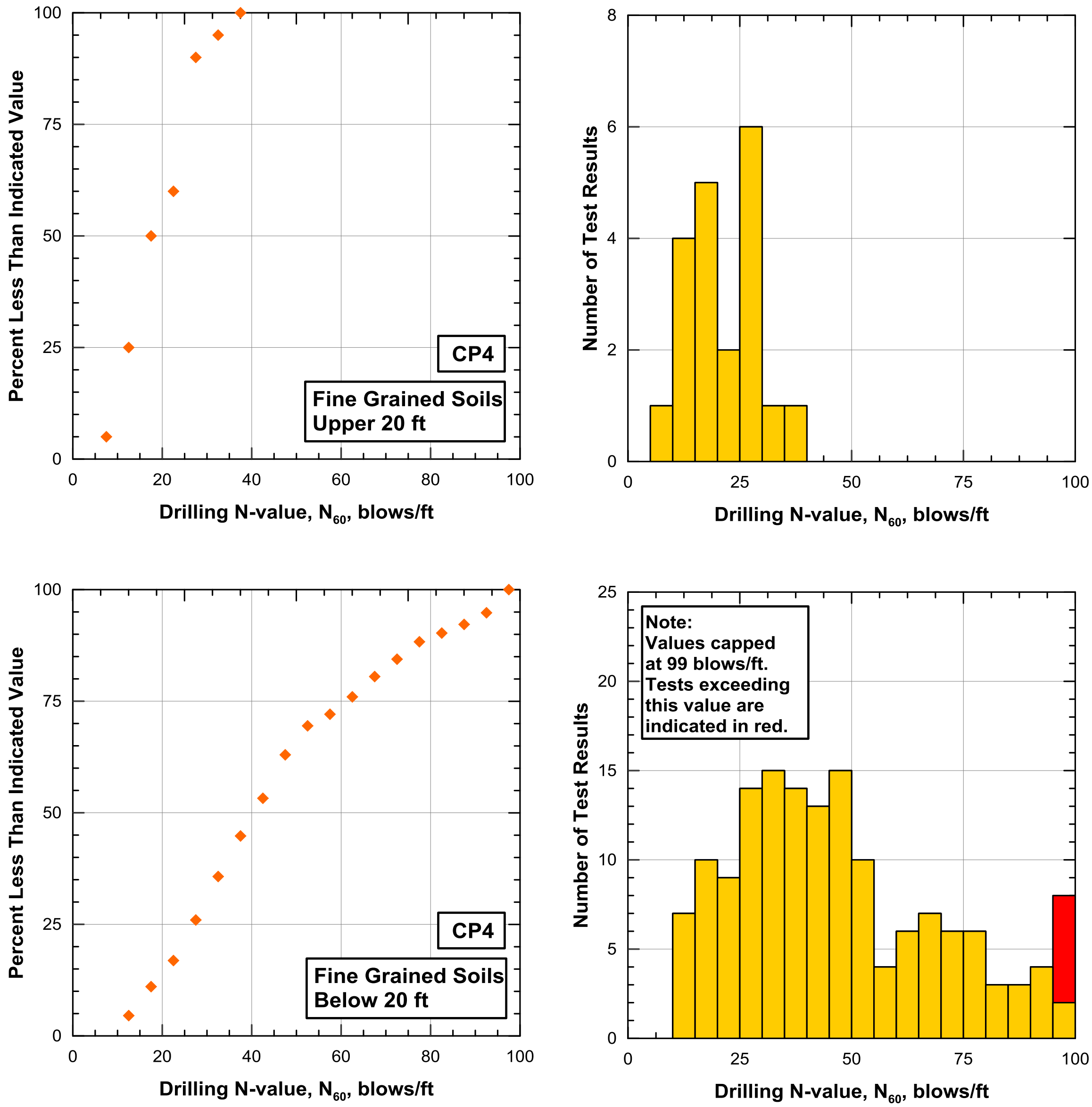

Figure A3.4-4
Statistical Summary of SPT N_{60} for Fine Grained Soils – CP4

A3.5 Cone Tip Resistance

Table A3.5-1
Statistical Summary of Cone Tip Resistance – CP4

Cone Tip Resistance	CPT			
	Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	1035	16382	2780	10636
Mean, tsf	39	96	93	307
Median, tsf	29	47	91	223
Standard Deviation, tsf	26	62	62	138
Maximum, tsf	233	476	420	771
Minimum, tsf	5	7	9	27

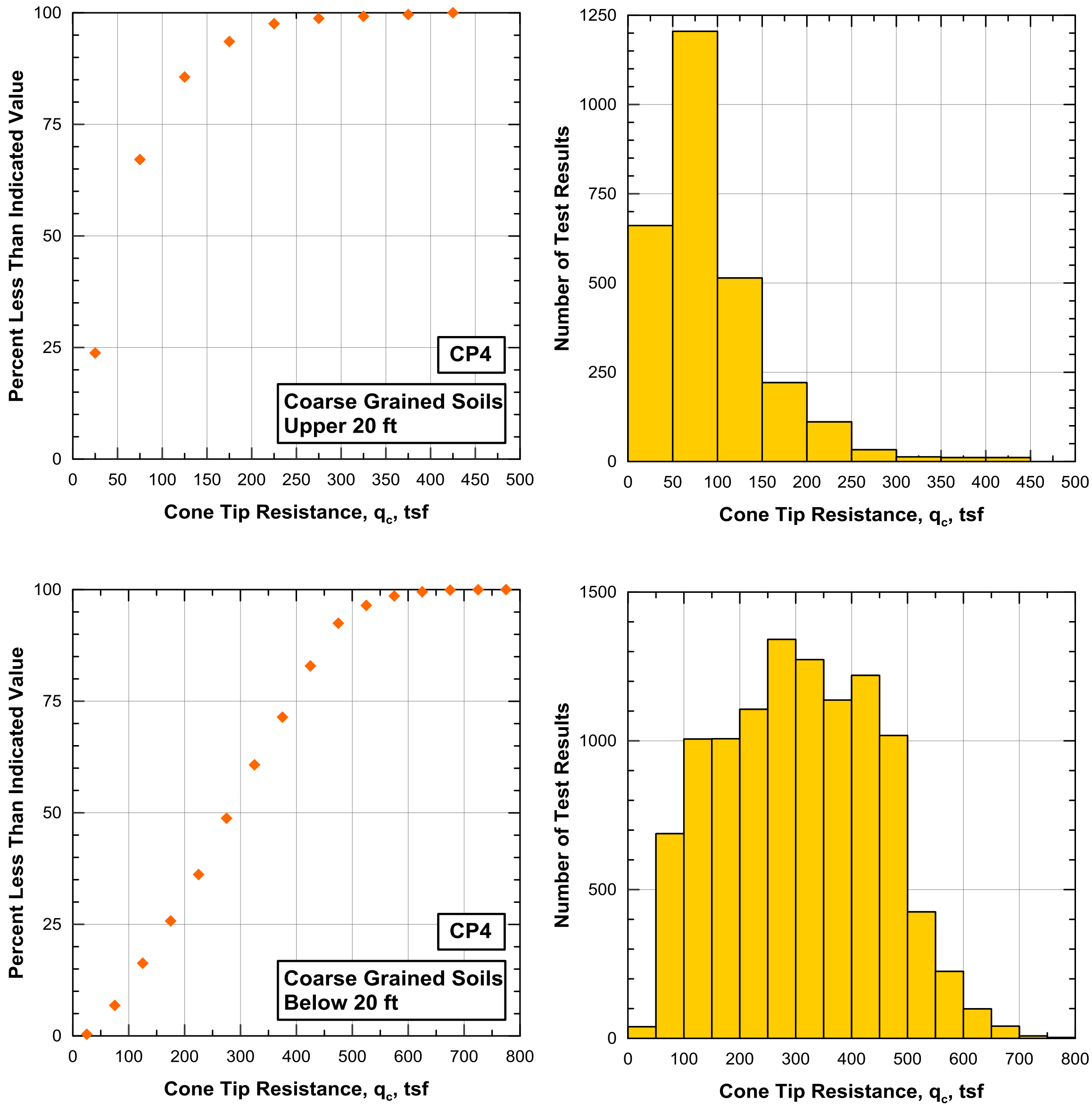

Figure A3.5-1

Statistical Summary of Cone Tip Resistance from CPT Data for Coarse Grained Soils – CP4

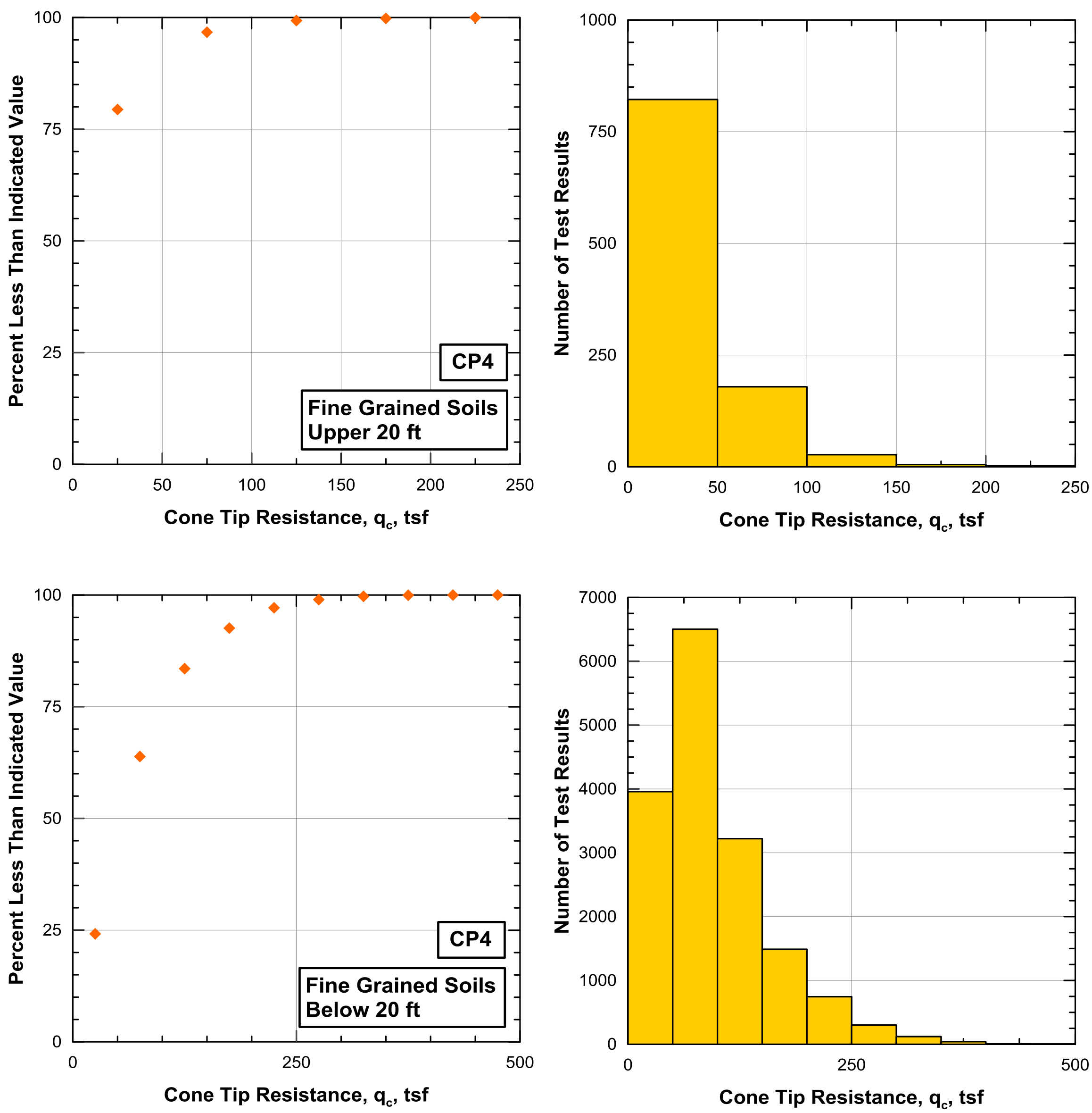

Figure A3.5-2
Statistical Summary of Cone Tip Resistance from CPT Data for Fine Grained Soils – CP4

A3.6 Soil Modulus

Table A3.6-1
Statistical Summary of Soil Modulus Estimated from CPT– CP4

Soil Modulus	CPT			
	Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 205 ft
No. Tests	1039	16382	2780	10636
Mean, tsf	660	1325	371	1209
Median, tsf	567	1308	304	1219
Standard Deviation, tsf	393	561	247	521
Maximum, tsf	2000	2000	1679	2000
Minimum, tsf	65	88	35	107

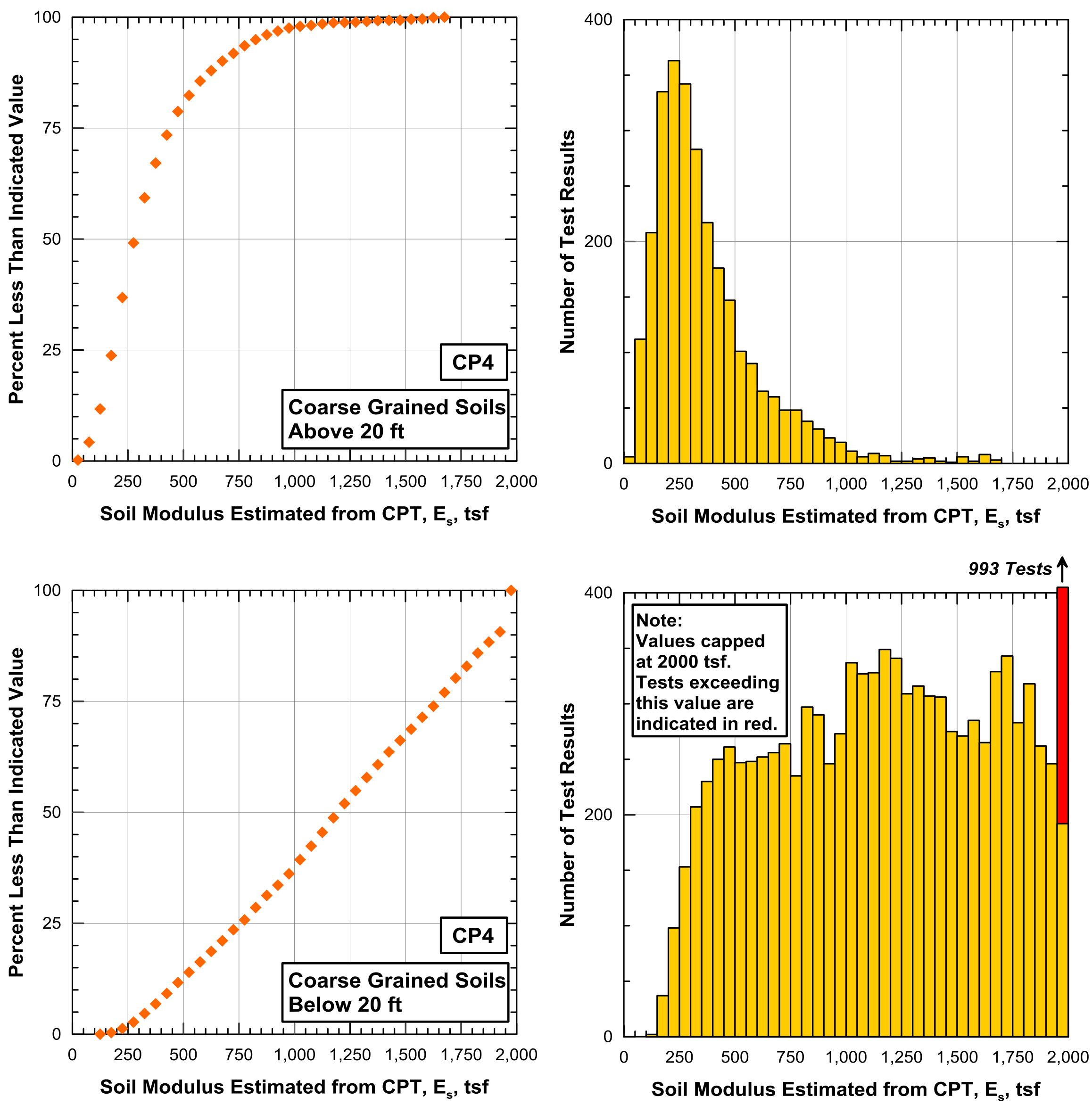

Figure A3.6-1
Statistical Summary of Soil Modulus Estimated from CPT Data for Coarse Grained Soils – CP4

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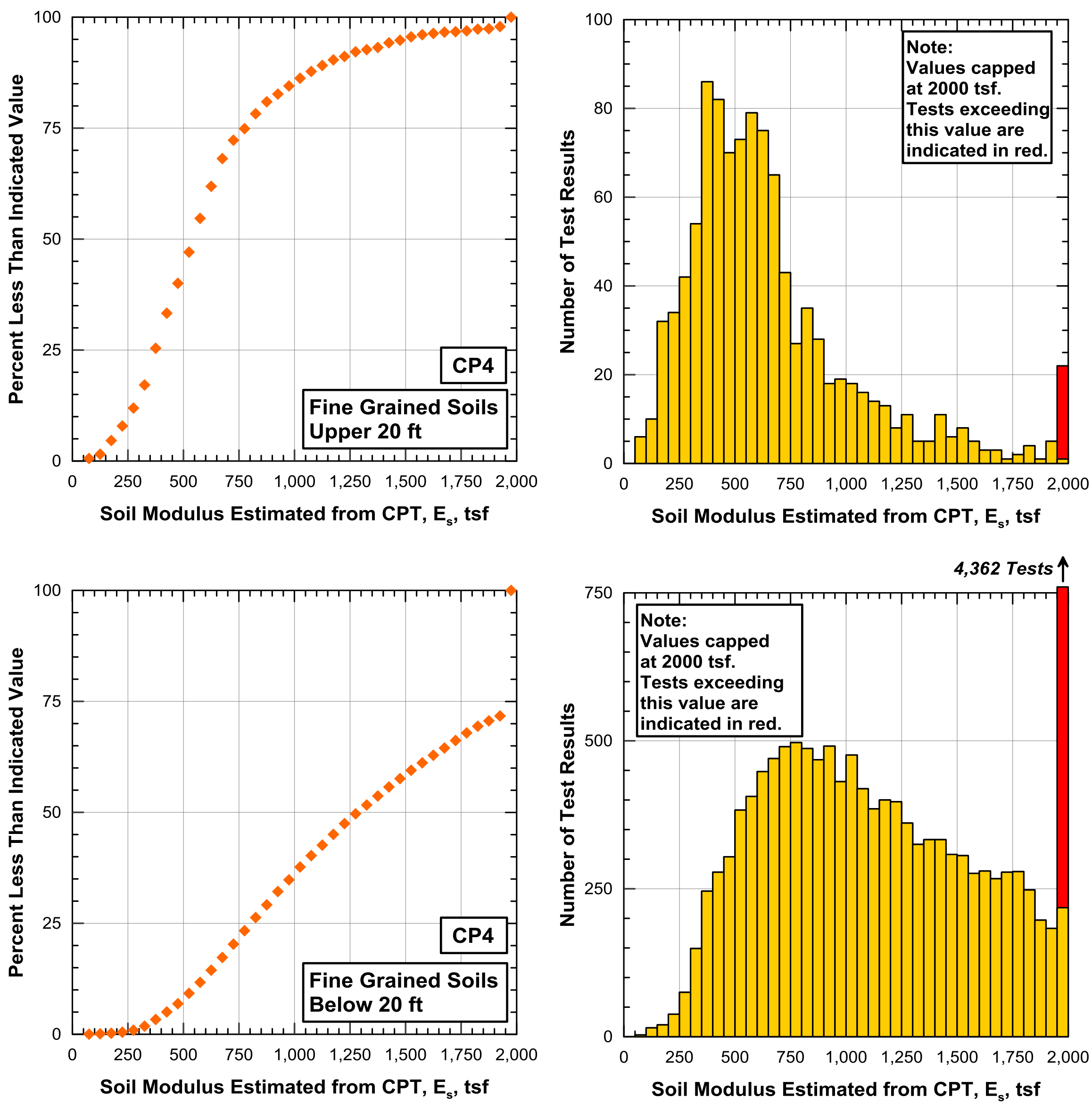

Figure A3.6-2
Statistical Summary of Soil Modulus Estimated from CPT Data for Fine Grained Soils – CP4

Table A3.6-2
Statistical Summary of Soil Modulus Estimated from Drilling – CP4

Soil Modulus	Drilling			
	Fine		Coarse	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	-	-	40	248
Mean, tsf	-	-	151	299
Median, tsf	-	-	140	271
Standard Deviation, tsf	-	-	81	129
Maximum, tsf	-	-	341	700
Minimum, tsf	-	-	42	111

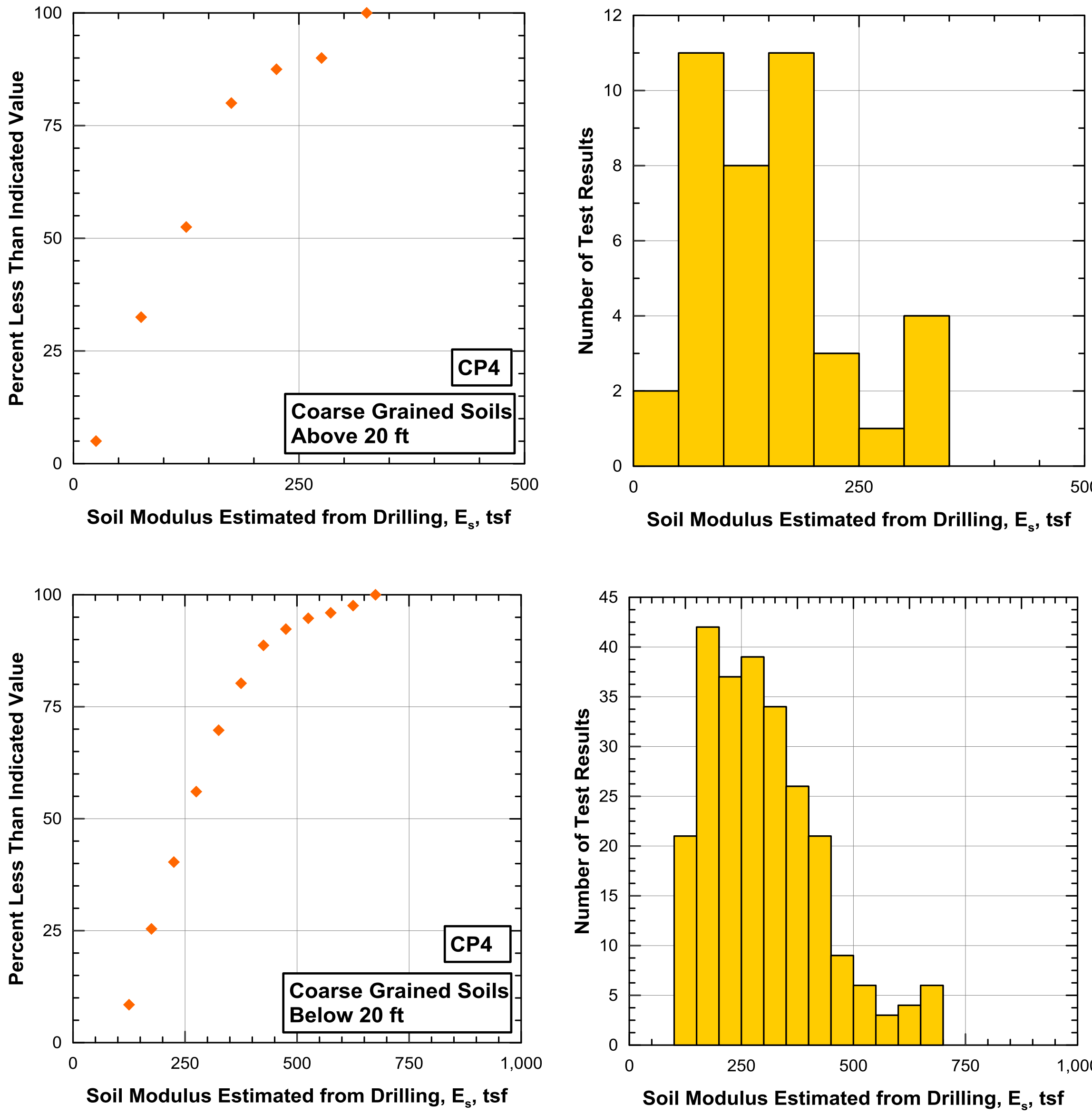

Figure A3.6-3
Statistical Summary of Soil Modulus Estimated from Drilling Data for Coarse Grained Soils – CP4

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A3.7 Undrained Shear Strength

Table A3.7-1
Statistical Summary of Soil Undrained Shear Strength from Laboratory Data – CP4

Shear Strength	Laboratory	
	Fine	
	Upper 20 ft	Below 20 ft
No. Tests	6	20
Mean, psf	2862	3072
Median, psf	2941	2744
Standard Deviation, psf	1336	1328
Maximum, psf	5000	5000
Minimum, psf	1097	1010

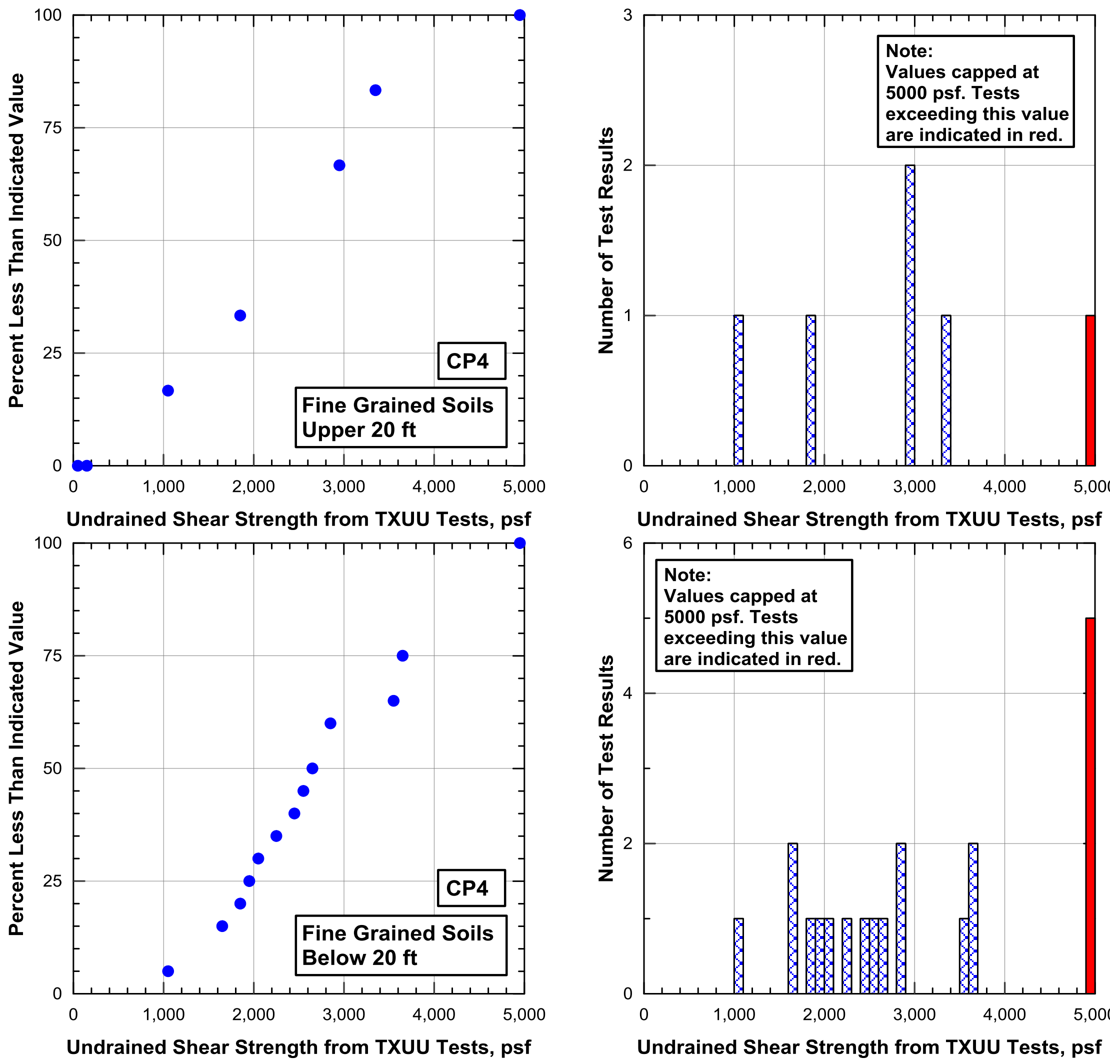

Figure A3.7-1
Statistical Summary of Undrained Shear Strength from Labratory Data for Fine Grained Soils – CP4

Table A3.7-2
Statistical Summary of Soil Undrained Shear Strength from CPT Data – CP4

Shear Strength	CPT	
	Fine	
	Upper 20 ft	Below 20 ft
No. Tests	1039	16382
Mean, psf	3632	4729
Median, psf	3783	5000
Standard Deviation, psf	1239	677
Maximum, psf	5000	5000
Minimum, psf	431	584

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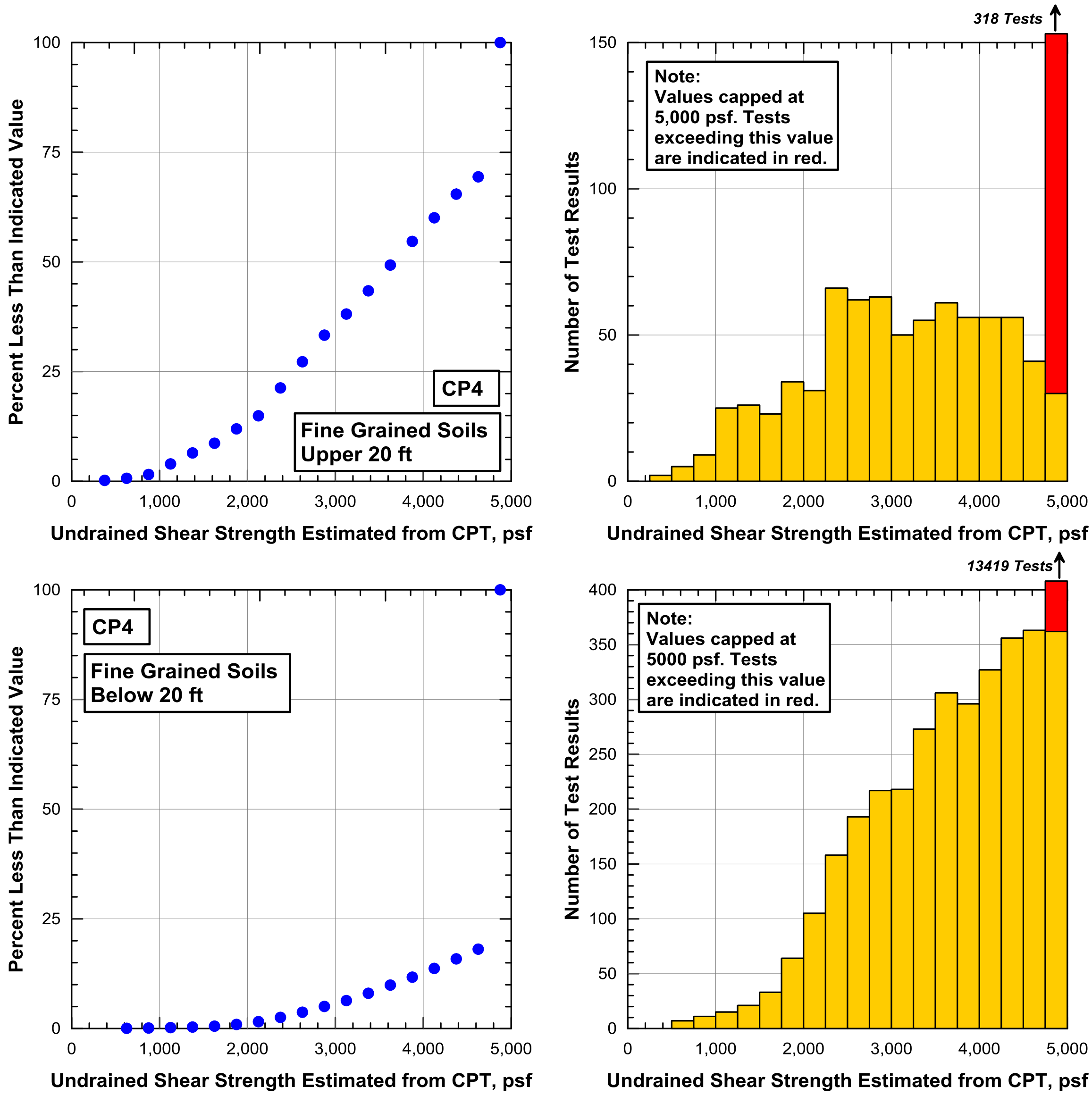

Figure A3.7-2
Statistical Summary of Undrained Shear Strength from CPT Data for Fine Grained Soils – CP4

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A3.8 Existing Fill and Near Surface Soils

Table A3.8-1
Statistical Summary of Values for Existing Fill and Near Surface Fill

		Fines Content	Maximum Dry Density ($\gamma_{d,max}$)	Optimum Moisture Content (w_o)	California Bearing Ratio	R-Value
COARSE	No. of Tests	13	16	16	4	15
	Min	17.4	119.3	6.5	6	10
	Max	54.4	133.4	11.2	33	66
	Average	39.6	128	8.8	13.5	36.1
	Median	40.9	128.3	9.1	7.5	32
	Std Deviation	10.4	3	1.3	13	20.5
FINE	No. of Tests	-	16	16	4	15
	Min	-	119.3	6.5	6	10
	Max	-	133.4	11.2	33	66
	Average	-	128	8.8	13.5	36.1
	Median	-	128.3	9.1	7.5	32
	Std Deviation	-	3	1.3	13	20.5

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A4.0 References

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